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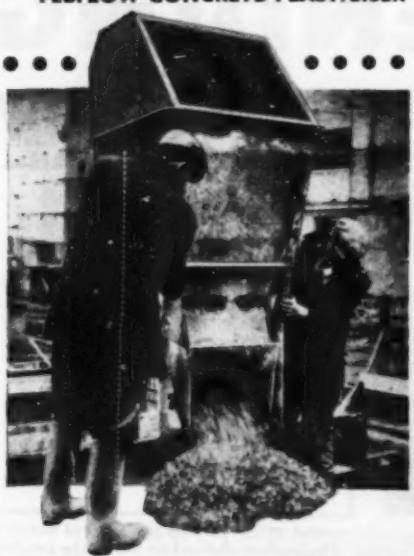

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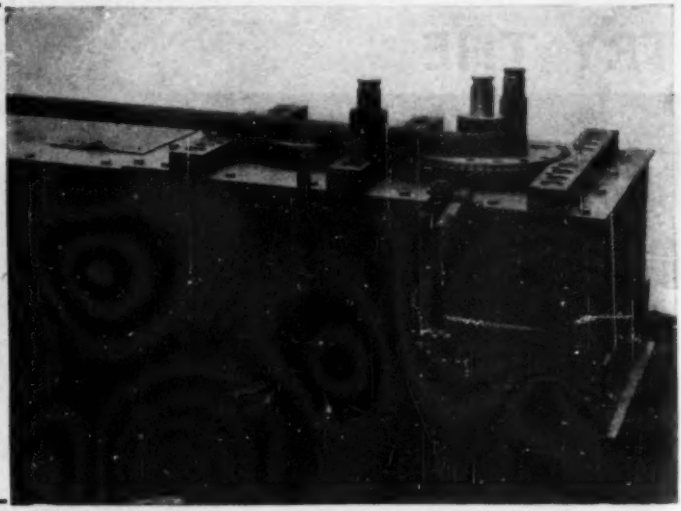

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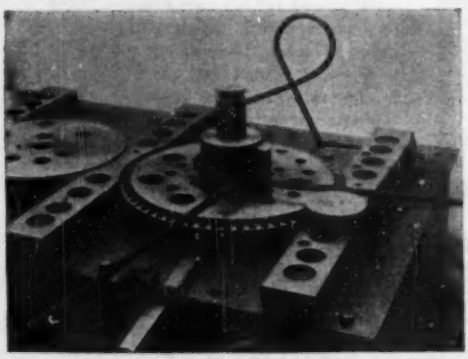

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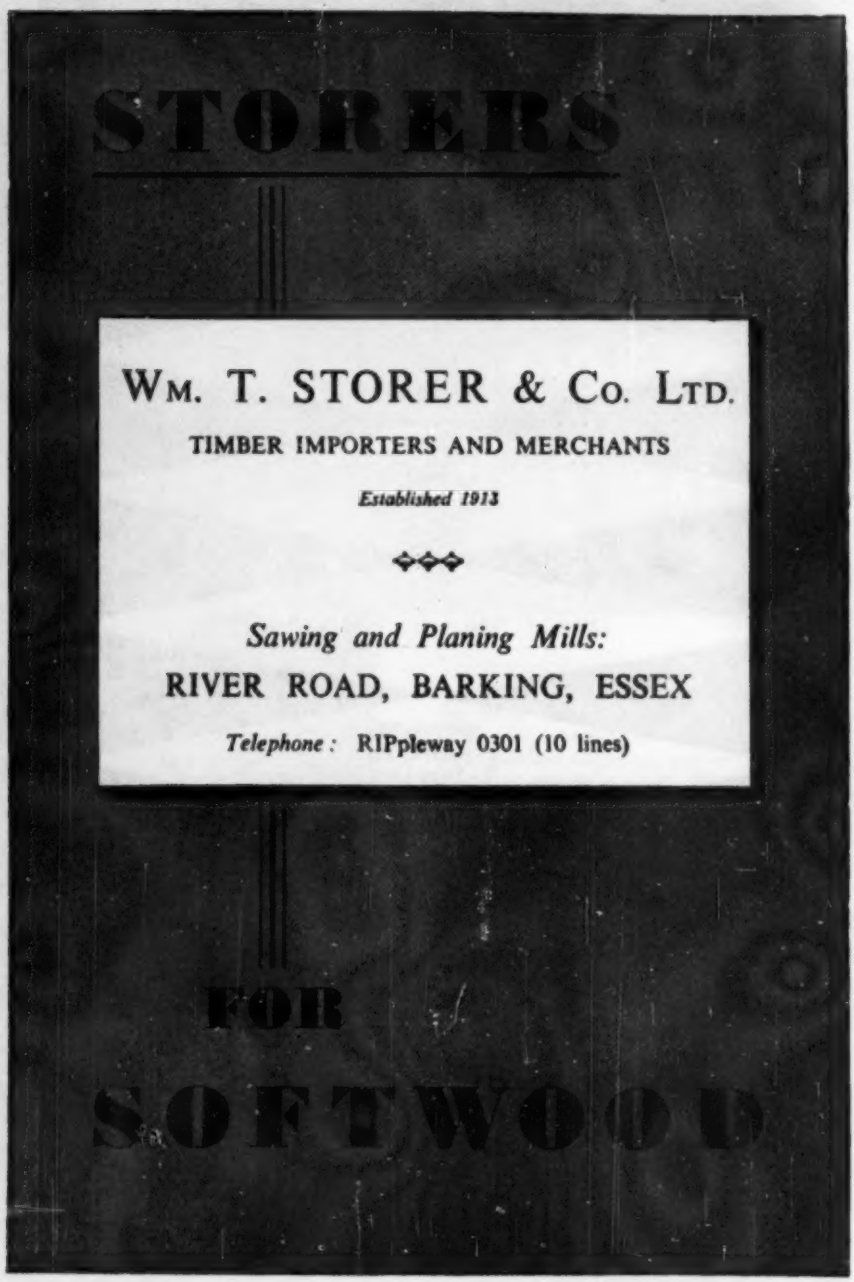
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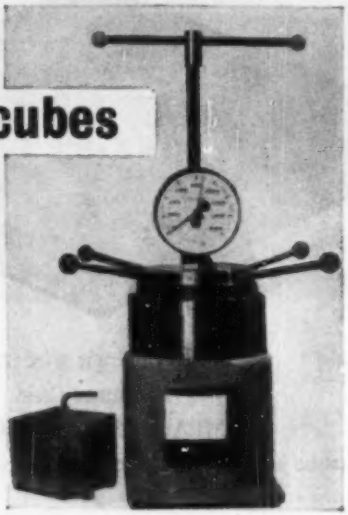

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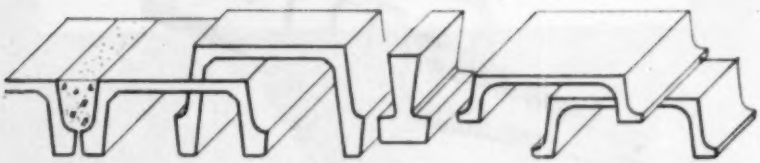

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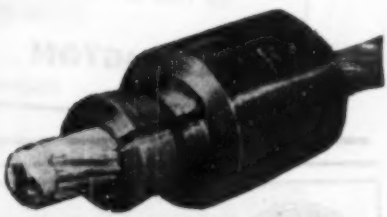

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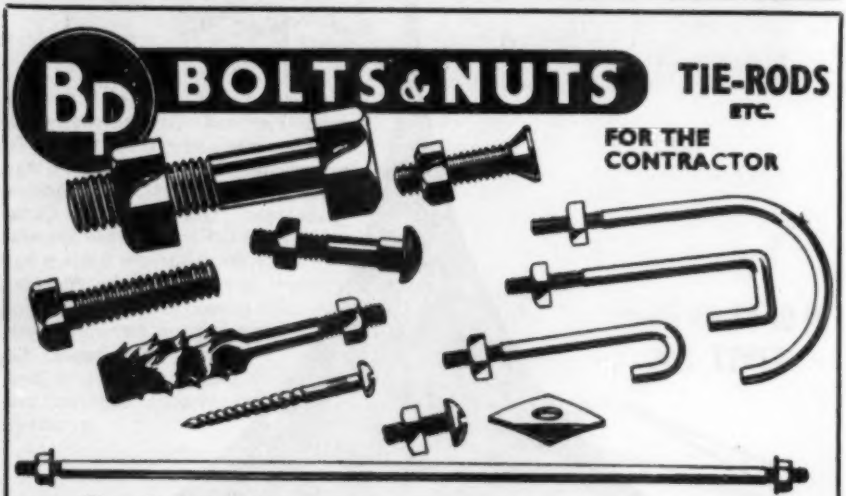

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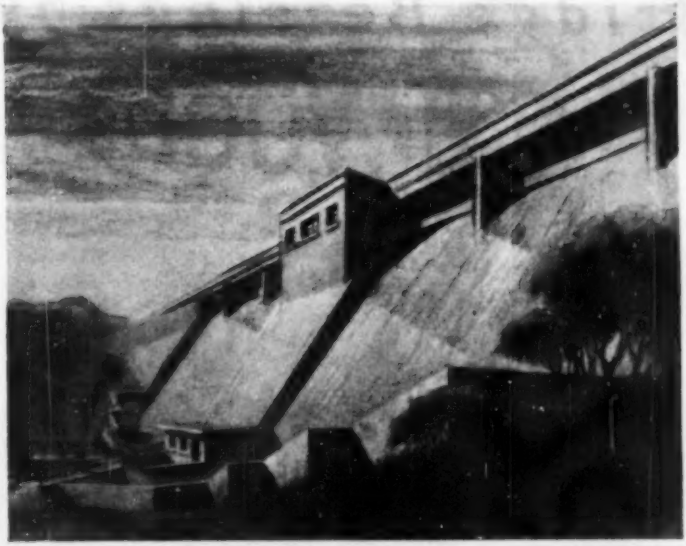

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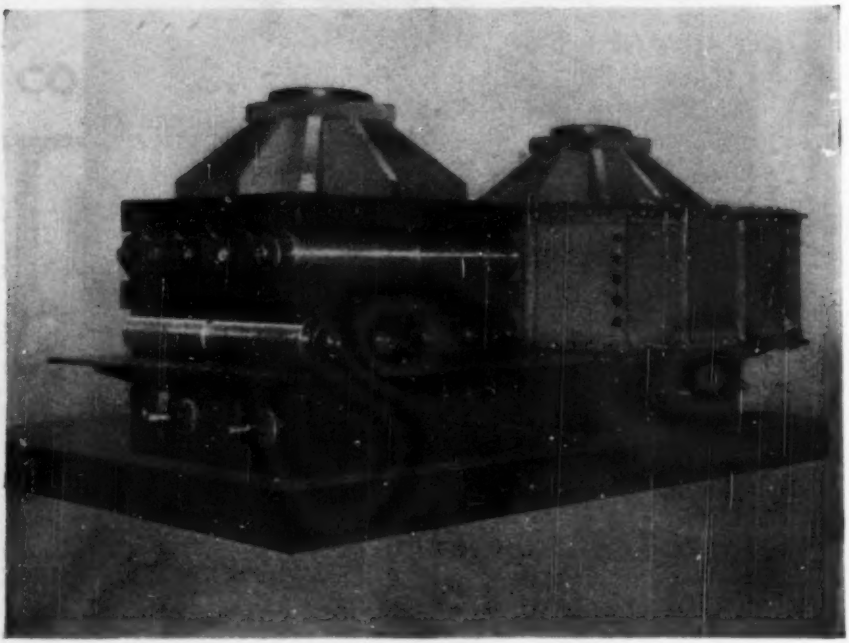

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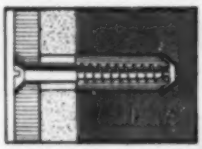

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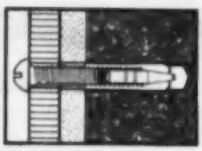
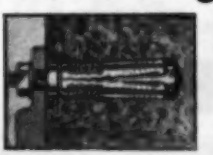
Rawlplug WHITE BRONZE PLUGS are specially suitable when the fixing is subject to very-high temperatures, such as the outer brick coverings of furnaces. Use also for underwater fixtures.



NEW! RAWLSETS, for ¼" and ⅜" Whit. screws, are metal shell sockets which grip by expansion. Ideal for fixing signs, stoves, metal windows, fluorescent light fittings, etc. Write for leaflet.

Bolt Fixings



For bolting heavy equipment, use RAWLBOLTS, a dry fixing that grips by expansion. No cold chiselling, no waiting for concrete to harden. Sizes up to 1" bolt diameter.

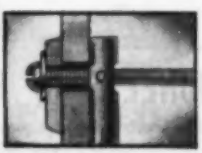
Use Rawlplug BOLT ANCHORS for bolting down in extremely wet or corrosive situations. Caulking completely seals the anchorage. Use also for anchoring Diesel Generators, Power Hammers, etc.

Rawlplug CEMENT-IN-SOCKETS are positioned before concrete is poured, eliminating hole-boring. They provide strong, permanent fixings wherever bolt locations can be pre-determined. For bolt diameters ½" to 1"

Cavity Fixings

Screwed up from the front the amazing RAWLNUT forms its own rivet head behind the material—airtight, watertight, vibration-proof, squeak-proof! For all thin or hollow materials.

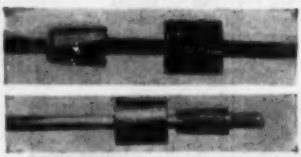
For ceilings of lath and plaster plasterboard, etc., use Rawlplug SPRING TOGGLES. The inserted Toggle springs open behind the material, spreading the load over a wide area.



Rawlplug GRAVITY TOGGLES are ideal for hollow materials from ⅜" thick. Inserted horizontally, the long end falls into correct position by gravity. The screw is then tightened in normal way. For ¼", ⅜" and ½" Whit. Screws.

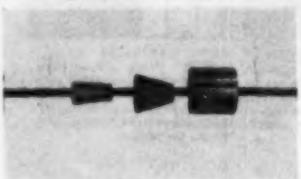
PSC for all prestressing equipment

P.S.C. PRE-TENSIONING



STANDARD MONOWIRE AND MONOSTRAND GRIPS

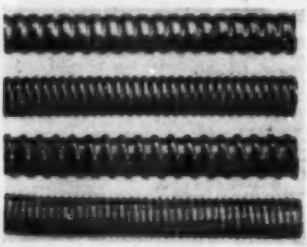
MULTI-USE MONOWIRE AND MONOSTRAND GRIPS



ADVANCED QUICK-RELEASE-TYPE MONOSTRAND GRIP

MONOSTRAND PRE-TENSIONING JACK

HYDRADUCT SHEATH



HYDRARIGID (normal)

HYDRARIGID (narrow corrugations)

HYDRARIGID (reverse corrugations)

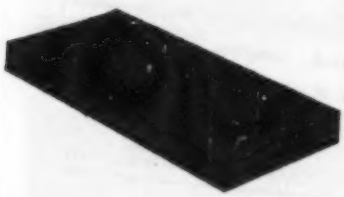
HYDRAFLEX

PIONEERS OF PRESTRESSED CONCRETE IN GREAT BRITAIN

Other prestressing equipment supplied by us includes Freyssinet Cone Anchorage, Freyssi-Strand Anchorage, Freyssinet Multi-Strand System, P.S.C. Monowire Anchorage, P.S.C. Monowire Tank Anchorage, P.S.C. Monostrand System, Grouting Equipment, Pre-tensioning Jacks, etc. Full details and complete technical information on prestressing equipment are always available on request.

PSC for FREYSSINET

RUBBER BRIDGE-BEARING PAD



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Ref.	Outside Diameter (inches)	Maximum Force (tons) at 2,000 lb/in.²
07	2.75	3
12	4.72	9
15	5.90	17
22	8.66	41
25	9.84	54
27	10.6	64
30	11.8	81
35	13.8	114
42	16.5	168
50	19.7	245
60	23.6	357
87	34.2	772
92	36.2	866

These hydraulic jacks of unusual and unconventional design permit enormous forces to be exerted where the movement required is small. They have considerable application in under-pinning, levelling, and thrust control in engineering structures.

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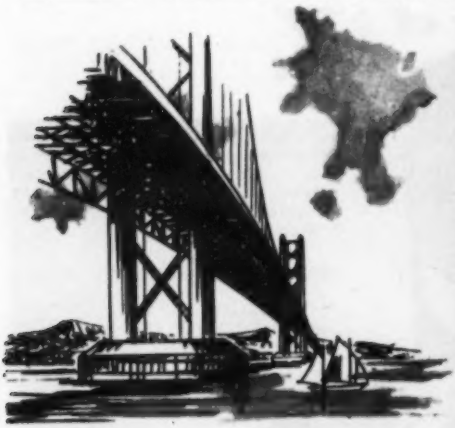
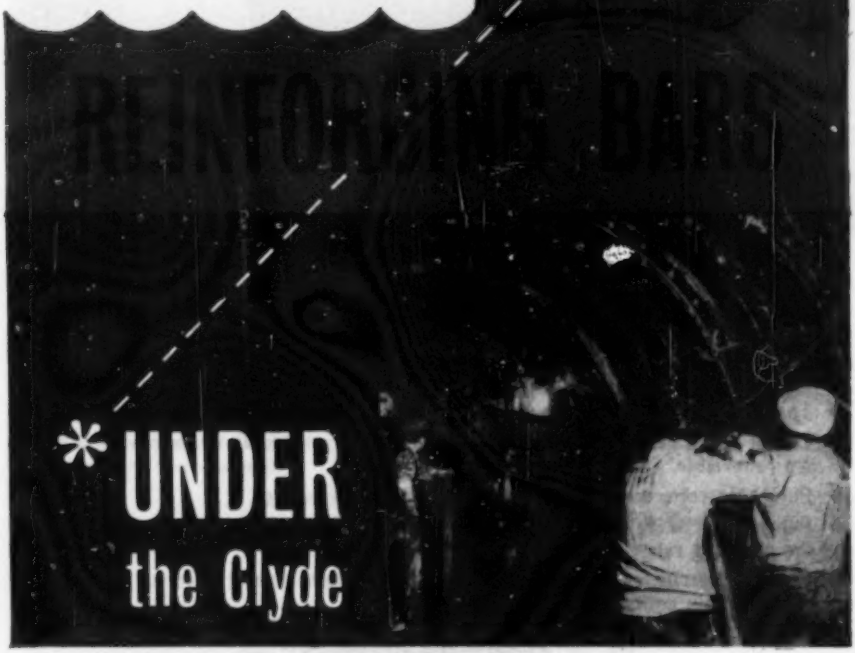


OVER * the Forth



* UNDER the Clyde

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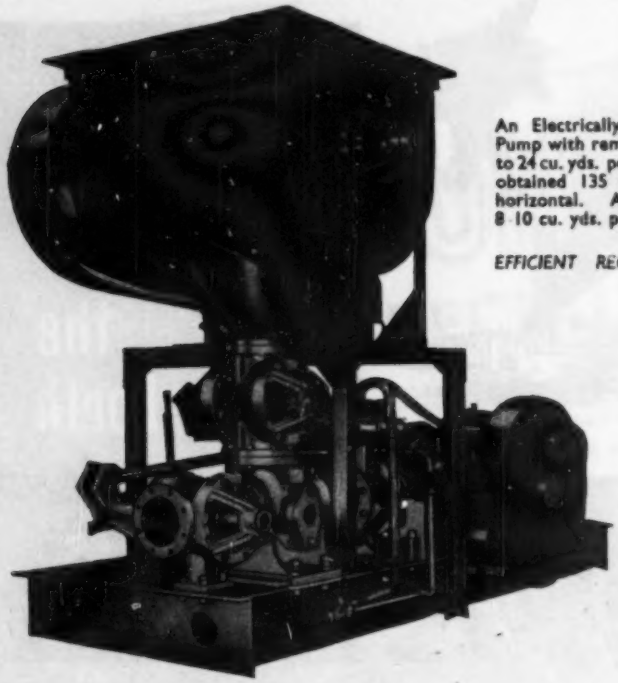

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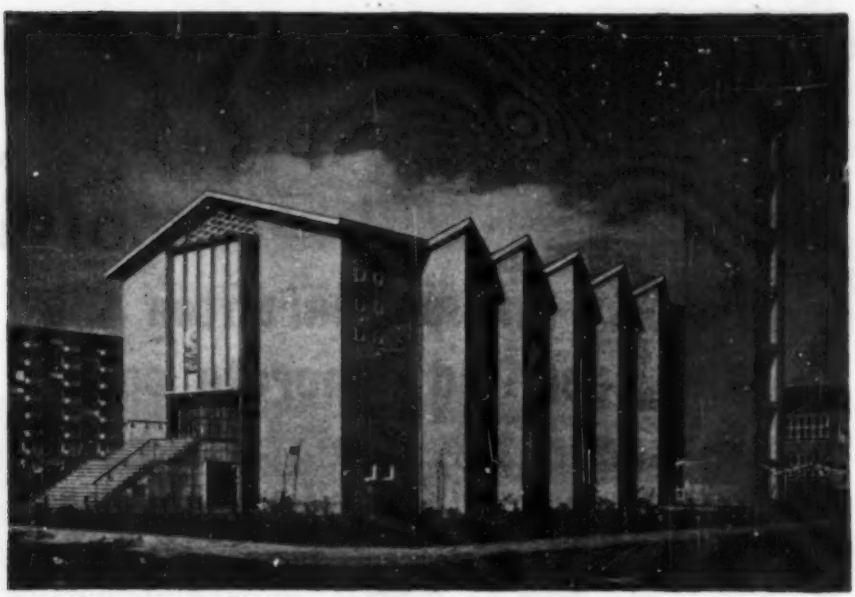

'This new church at Vlaardingen near Rotterdam is founded on fifty-nine 17-in. diameter Vibro piles 81 ft. long. The working load per pile is 68 tons, and 19-in. and 24-in. diameter shoes were used to develop the appropriate bearing capacity.

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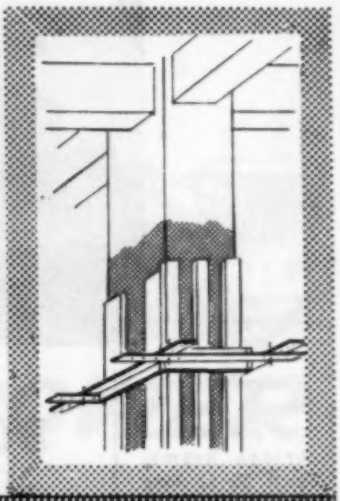

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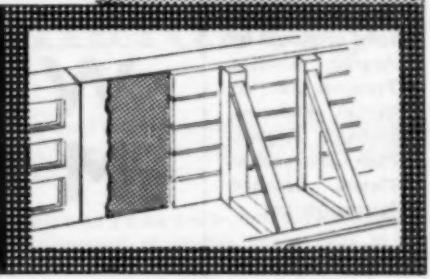

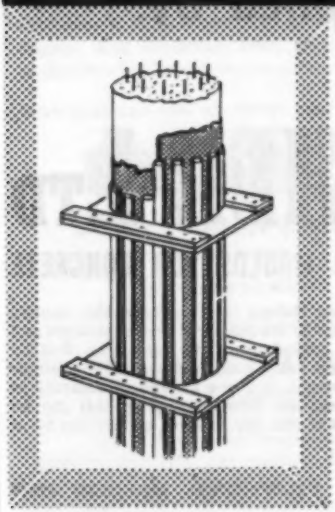

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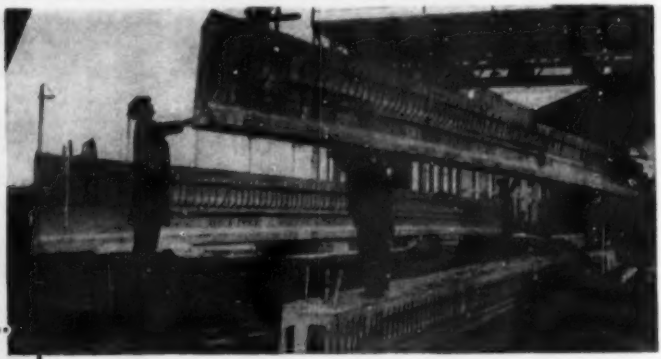

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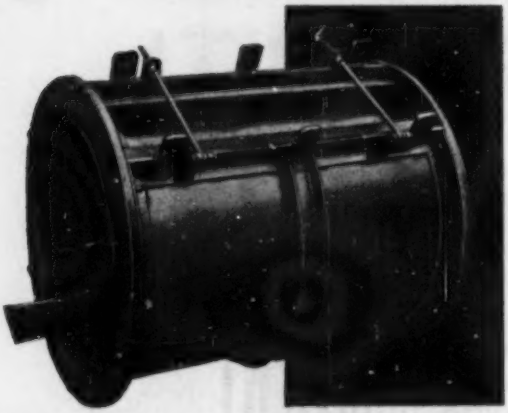

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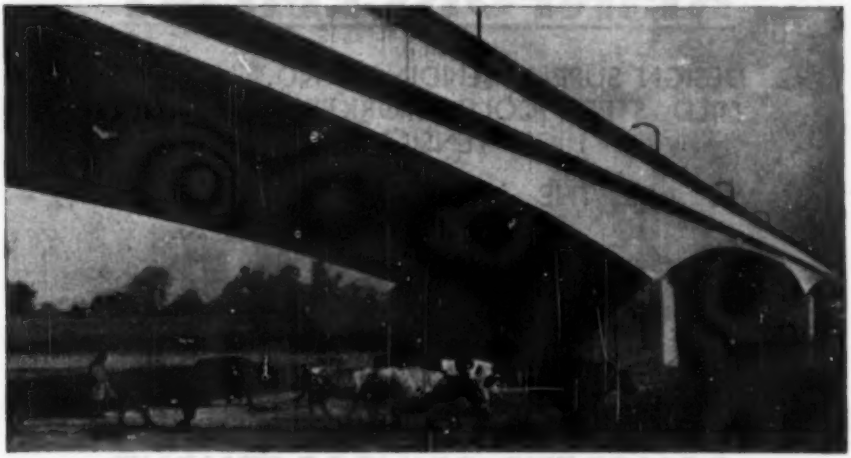

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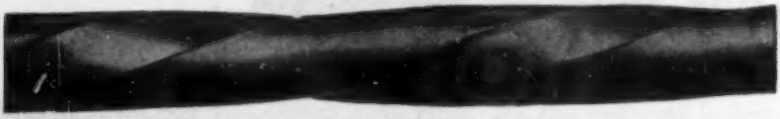

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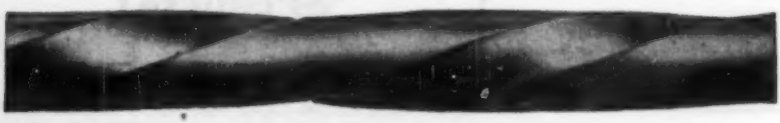

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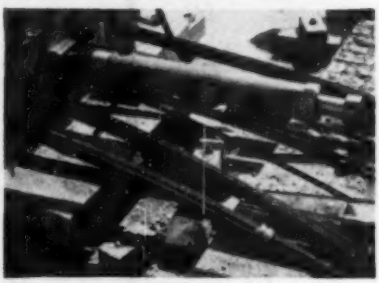

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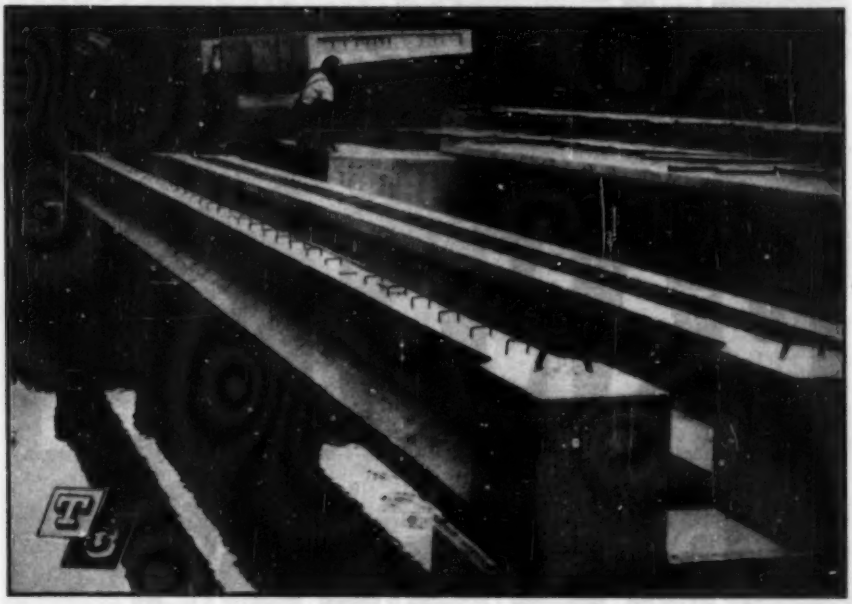

New road bridge during construction on A.50 at Fostons, Derbyshire, showing 40-ft. prestressed beams (pretensioned steel) designed to M.O.T. requirements. Main Contractors: D. T. Bullock & Co. Ltd., Aldridge, Staffs. By courtesy of the County Surveyor of Derbyshire—S. Mehew, Esq., O.B.E., B.Sc., M.I.C.E., A.M.T.P.I.

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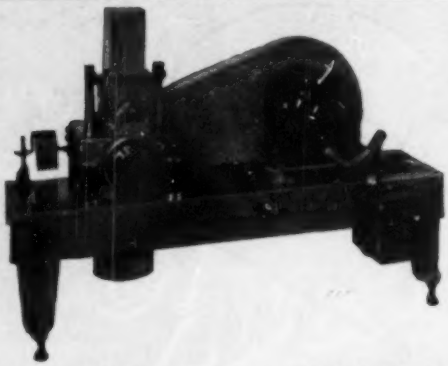

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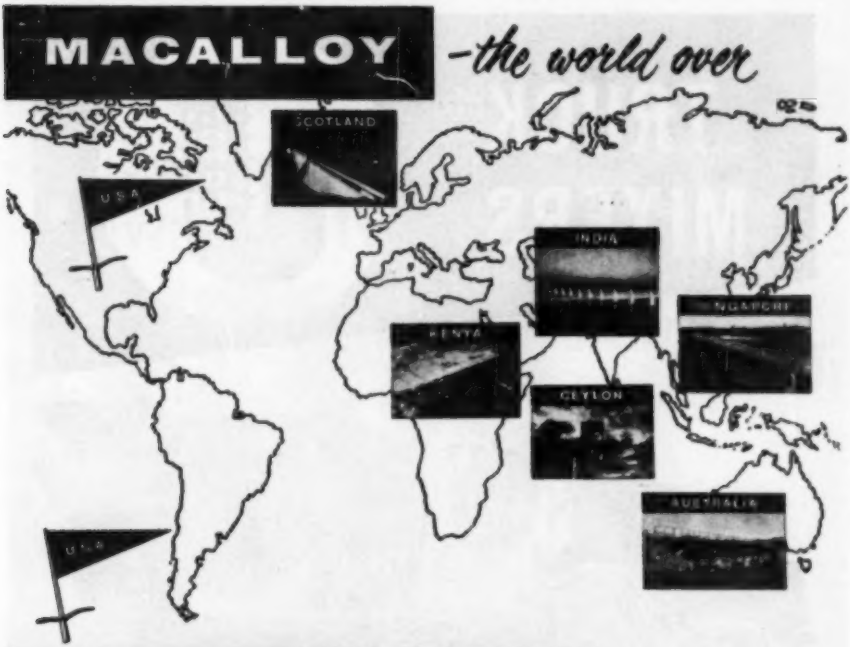

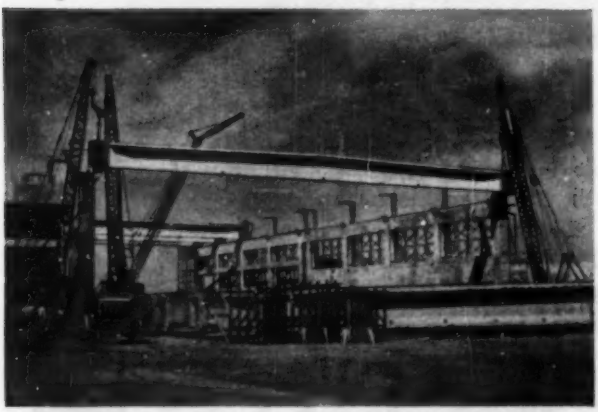

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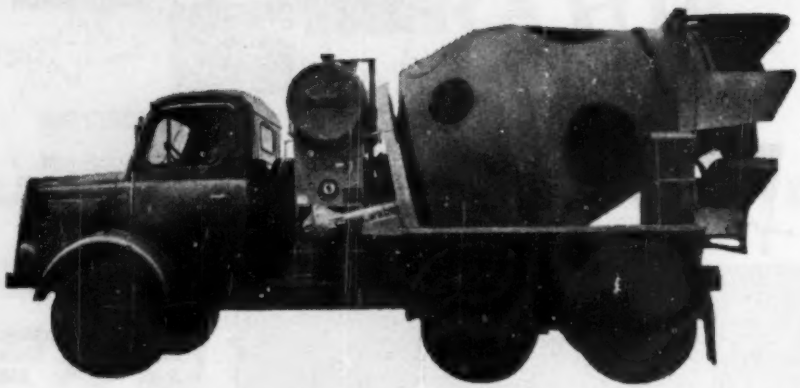

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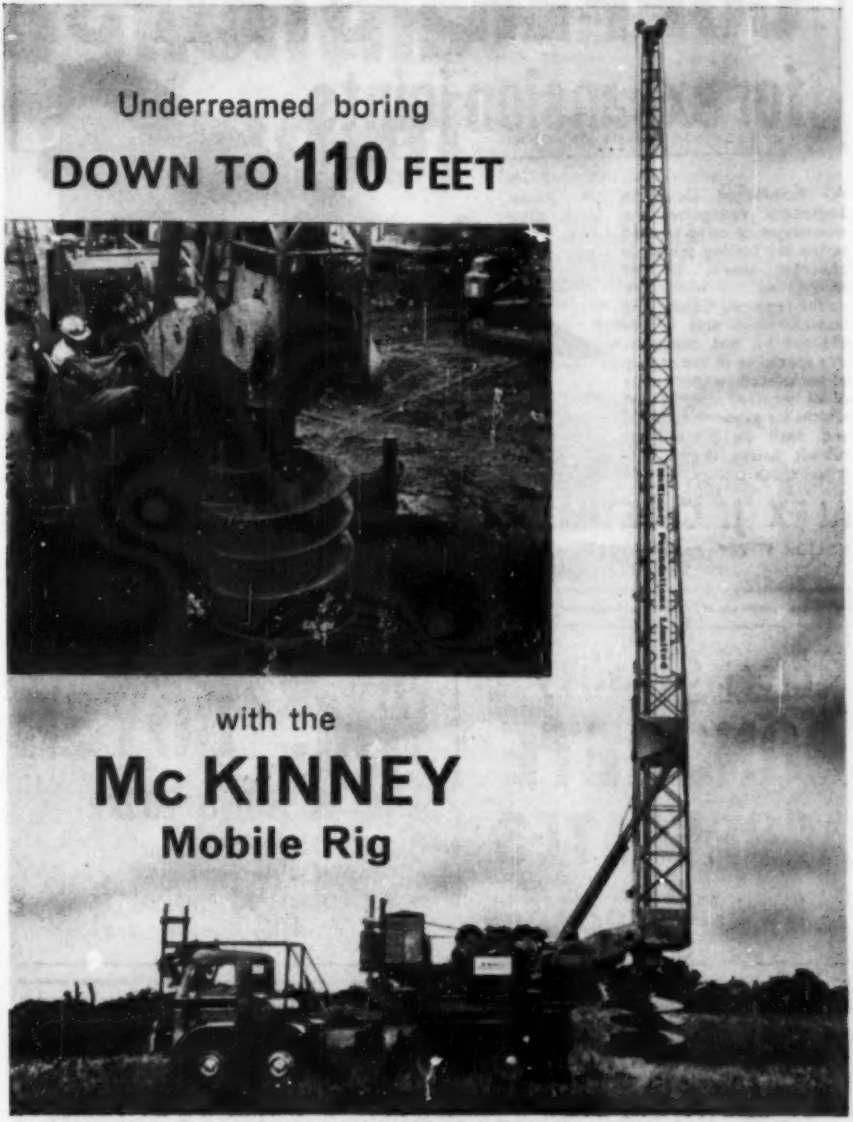

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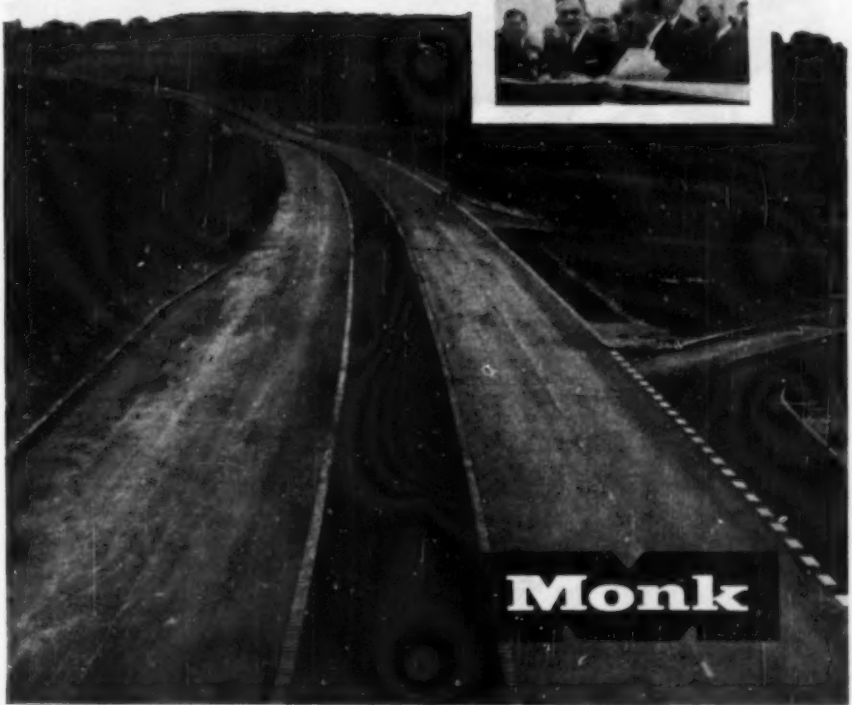

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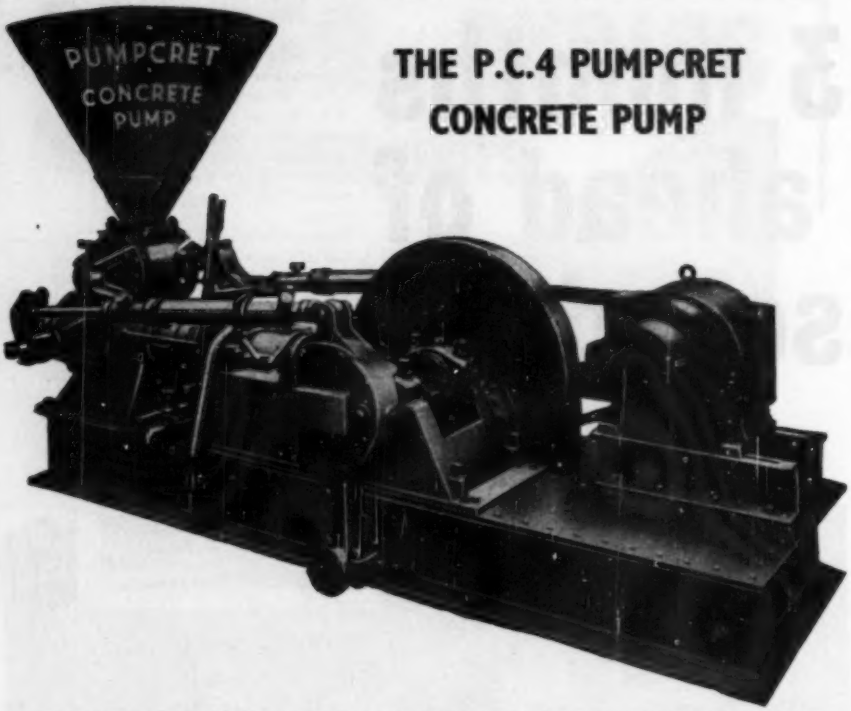

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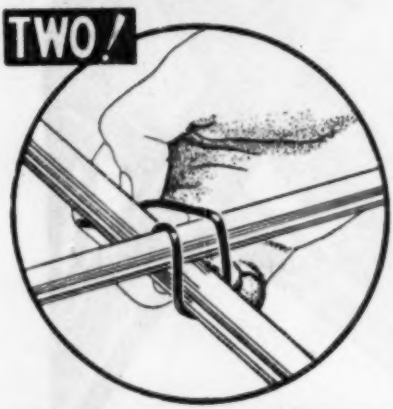

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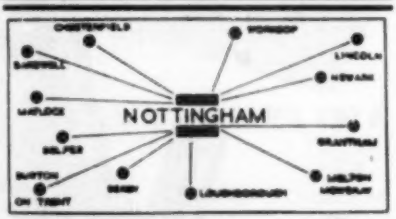

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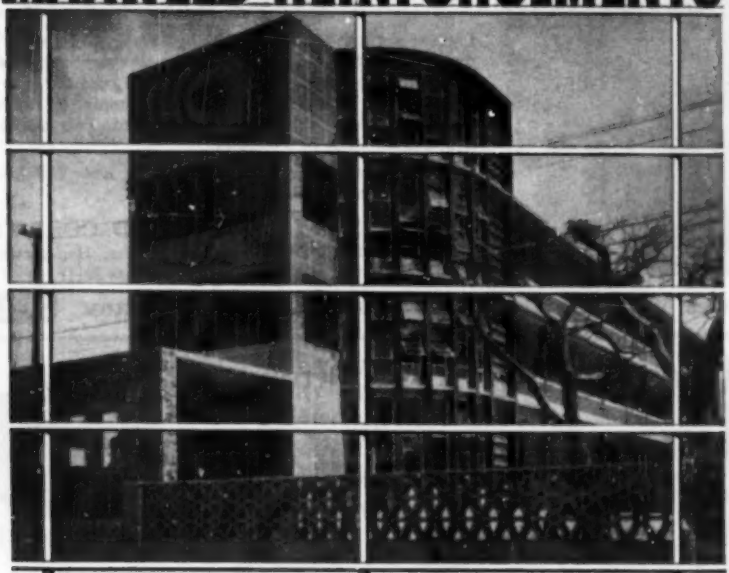

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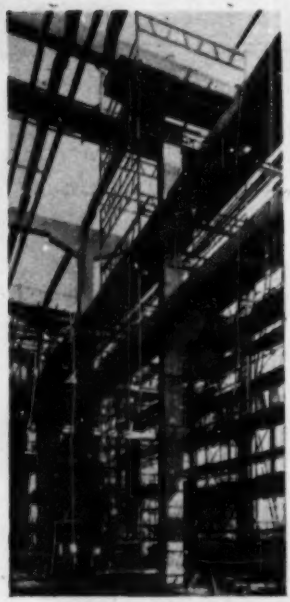

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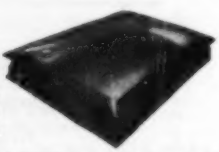

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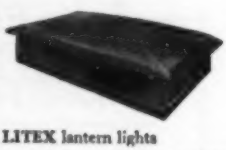

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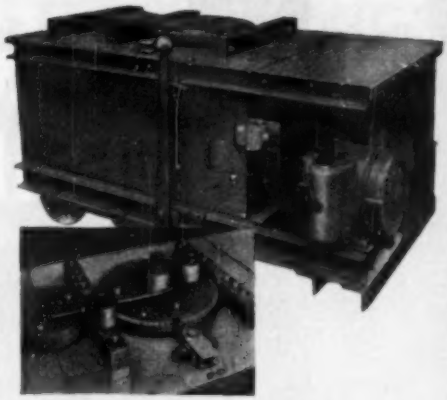

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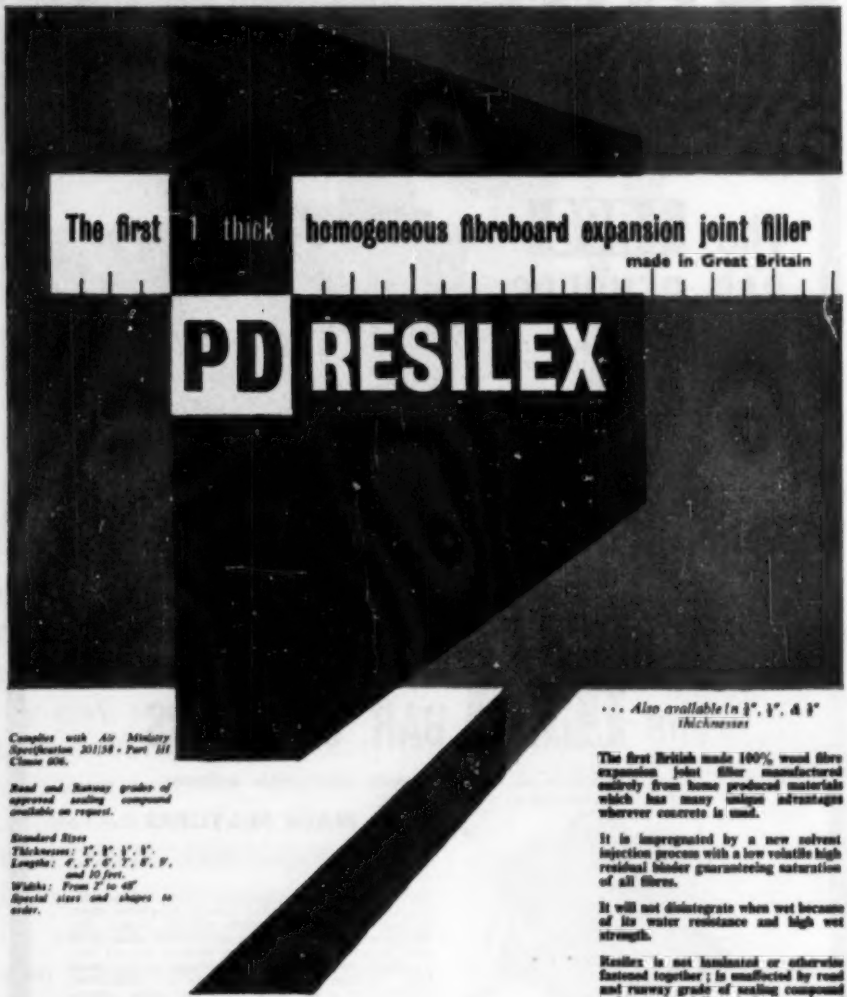

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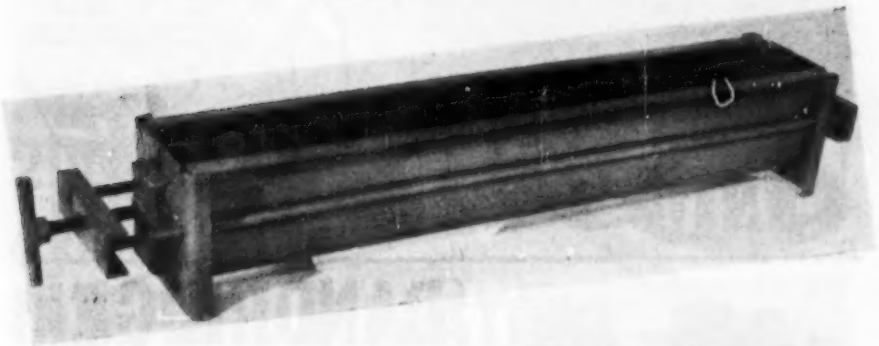

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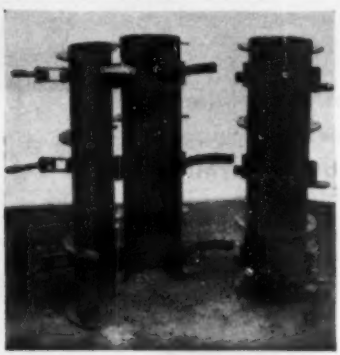

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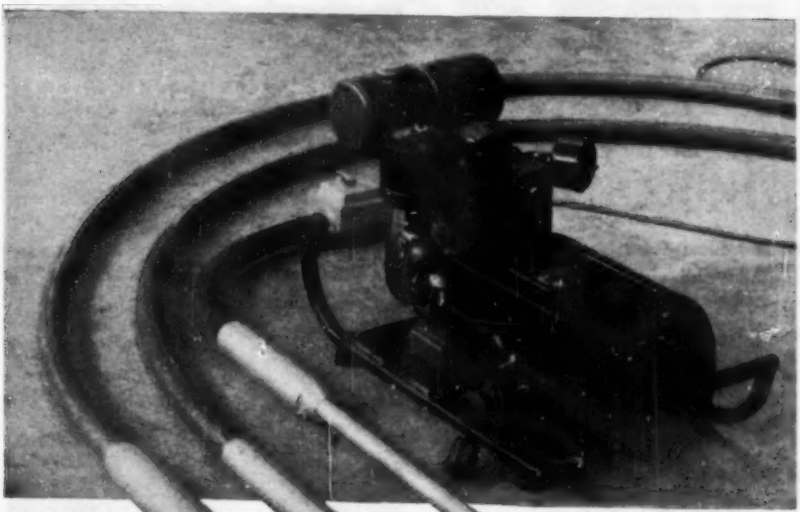

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING
INCLUDING PRESTRESSED CONCRETE

Volume LV, No. 12. LONDON, DECEMBER, 1960.

EDITORIAL NOTES

Radioactive Isotopes in the Concrete Industry.

THE civil engineering, building and allied industries are often thought to be backward in adopting new ideas and developments. This is hardly true in the narrow field of the use of radioactive isotopes. Isotopes are forms of an element having the same chemical properties, but differing in the number of neutrons contained in their atomic nuclei. A radioactive isotope is one which is radioactive naturally, or has been made radioactive by changing the number of neutrons in the nucleus by irradiation in an atomic pile or by bombardment in a cyclotron. Radioactive isotopes used as sources of radiation are becoming increasingly common and are applied in many industrial activities, and have been or can be used in the construction and testing of concrete structures. They are in general use in industry to measure or test properties of materials or to detect the presence of constituents not visible to the eye or readily detectable in other ways.

The density of the concrete in a structural member can be determined by applying a source of radiation close to the surface of the member and measuring the radiation reflected. The stream of radiation penetrates the concrete and collides with, and is reflected from, the atoms of the materials of which the concrete consists. The greater the number of atoms the greater the amount of radiation reflected from the atoms, and the amount so reflected is therefore a measure of the density of the concrete or of the thickness of a piece of concrete. The operation of some instruments depends on the property of materials to reflect radiation. Other instruments containing radioactive isotopes are used in radiography in place of X-ray equipment which may be unwieldy and unable to be used in restricted positions. An apparatus utilising radioactive isotopes is small, generally portable, and independent of supplies of electricity or water.

Instruments employing radioactive isotopes are particularly advantageous in research and testing because of the ability of radiation to penetrate solids, and provide information continuously about the interior, without destroying the sample. A high degree of accuracy is possible and much smaller quantities of radioactive isotopes can be measured or detected than is possible by chemical analysis. The accuracy of some of the instruments using radiation is such that a film of material only fifteen atoms thick can be measured.

Apparatus employing radioactive isotopes has been used to determine the position of the reinforcement in concrete structures, and to examine the efficiency of welds and of grouting in cable ducts in prestressed members. The use of radiography to detect defective grouting is the subject of a report entitled " The Application of Gamma Radiography to Concrete " by J. A. Forrester and issued by the Cement and Concrete Association. The source of radiation for the experiments described in this report was a cylinder of radioactive cobalt (^{60}Co) of 275 millicuries strength which was placed 18 in. from a sensitive film. The thicknesses of the concrete members were 5 in. and 6 in. requiring exposure for 55 minutes and 65 minutes respectively. The resulting pictures indicate clearly the parts of the duct where there is little or no grout.

In other instruments, radiation is used to measure the moisture content of soil at the surface or within narrow boreholes. Similar instruments can be used to measure the water content of concrete. The accurate alignment of points on opposite sides of concrete walls and other slabs is possible, as is done in the case of steel plates in the ship-building industry, by passing a beam of radiation through the slab from a point on one face ; a detector at the other face indicates the point at which the beam leaves the wall and so establishes the position of the point on the hidden face.

A property of the radioactivity of an isotope of which much use is made is its indestructibility by physical means. The radioactivity is not affected by heat, pressure, vibration, dampness or such phenomena. Because of this, radioactive isotopes may be mixed with materials in industrial processes to trace the progress of the materials through various stages of processing such as firing the raw materials of cement in a kiln, or crushing and grinding in a mill. The efficiency of mixers can be tested by including radioactive isotopes in the materials and measuring the dispersion of the isotopes. The exact time required to mix materials sufficiently can be determined and might save prolonged mixing of concrete, but it is doubtful whether the cost of such an investigation in practice would be worth while. Another example of this application is the tracing of the passage of cement grout through a duct or in a joint. A small quantity of a radioactive isotope is mixed with the grout which is then pumped in the usual way into the duct or joint. The progress of the grout is traced by means of a detector. Since the " half-life " of the radioactive isotope used for such a purpose is only a few hours there is said to be no lasting danger.

In common with some other industrial operations the use of radioactive material is accompanied by certain hazards, but as with other risks, these may be overcome by taking reasonable precautions. Radioactive isotopes can be divided into two classes from the point of view of safety. The first comprises sealed sources of radiation the risk of which is external radiation. The second comprises unsealed sources the danger of which is due to inhalation, ingestion or skin absorption of radioactive material, and constitutes external and internal radiation hazards.

The protection against external radiation is by shielding or distance. Methods of protection against internal radiation are similar to those of other industrial toxic materials. In each case the aim is to ensure that a person is subjected to as little radiation as possible and that the amount received never exceeds the amount regarded by the International Commission on Radiological

Protection as "not expected to cause appreciable bodily injury to a person at any time during his lifetime".

The United Kingdom Atomic Energy Authority is interested in increasing the use of radioactive isotopes in industry in this country and has established an advisory service to assist those who are interested in the use of these substances, to determine the most suitable isotope and to devise new means of application. Instruments designed by the Authority are available for industrial users.

Acoustics and Building.

THE British Standards Institution continues to pour out codes of practice and in the past twelve months or so there have been at least five of interest to the building and civil engineering industries. The latest publication* deals with noise abatement as related to the planning and construction of buildings. It is a matter of some seriousness, and one that raises a problem which is not easily solved, that the increasing intensity of noise in many facets of life to-day and the currently decreasing weights of components and cladding of modern buildings are opposing tendencies, because the sound-insulating properties of a structure decrease as the weight decreases.

It is important to differentiate between sound absorption and sound insulation. A material is a good absorber of sound if it has the ability to absorb and not to reflect sound; lining a room with a good sound-absorbing material will reduce the amount of noise within the room, but may not affect the amount of noise passing out of the room through the walls and other components. In general, lightweight acoustic tiles are sound absorbers but do not provide a barrier to the passage of sound. A material which is a good insulator of sound is one which provides such a barrier, and the best and cheapest insulators are heavy structural materials; in this respect ordinary concrete is rated highly. Attempts to make concrete of exceptionally great weight, such as by using steel shot as aggregate, generally result in a more expensive material, although of high sound-insulating value, and the extra cost is only justified in special cases such as the construction of sound-proof laboratories as has been done at the Building Research Station, where a weight of about 3 cwt. per cubic foot of concrete is obtained.

The Code comprises over a hundred pages into which is packed much information on the subject of acoustics. The contents vary from generalisations, such as that noise is more of a nuisance at night than during the day, to tables of useful practical data on the noise-reducing value of various types of building components. From these data it is seen that walls and partitions of clinker blocks, dense-concrete blocks, or ordinary concrete, and floors of concrete are superior to most other materials or combination of materials as regards sound-reducing value.

That engineers and architects require training in the subject of acoustics in building is borne out by experience, and notably by the example of a failure in this respect of a new and expensive school, a report on which was given recently in the press. Whether application of the recommendations of the Code will completely solve most problems of this nature is questionable, but it cannot be said that engineers and architects have not now available a guide to how a problem should be tackled in principle if not in all cases in detail.

* British Standard Code of Practice CP 3, Code of Basic Data for the Design of Buildings: Chapter III (1960). "Sound Insulation and Noise Reduction." (Obtainable from the British Standards Institution. Price 10s.)

Book Reviews.

"A Course of Reinforced Concrete Design." By T. J. Bray. (London: Chapman & Hall, Ltd. 1960. Third edition. Price 63*s*.)

THE text of the new edition of this well-presented book has been rewritten where necessary to conform to the British Standard Code of Practice No. 114 (1957), but in the main the subject matter is in the same form as in the previous edition. The treatment of simple problems, such as slabs spanning in one direction, is fairly comprehensive. Other subjects, such as slabs spanning in two directions, combined bending and compression, and arches, are dealt with in a more superficial manner. Most of the calculations are based on the modular-ratio method of design; the load-factor method is dealt with briefly and appears to be included as an afterthought. The book, which is well illustrated, is recommended for those requiring an elementary treatment of the principal features of reinforced concrete design but, to this reviewer, the book seems to waver between an attempt to be a students' text-book and the provision of a set of design charts for detailers.

"Data Book for Civil Engineers. Vol. 1. —Design." By Elwyn E. Seelye. (London: Chapman & Hall, Ltd. Third edition, 1960. Price £9 12*s*.)

THE third edition of the first volume of a three-volume work follows the large format and tabular presentation of the previous editions, but much new matter, especially in the sections on concrete, roads, airports and bridges, is incorporated. Where the data are regional, they apply mainly to practice in the U.S.A., but otherwise the information, particularly that dealing with framed structures, is of general application.

"Praktische Schalenstatik," Vol. 1. By J. Born. (Berlin: Wilhelm Ernst & Sohn. 1960. Price 50 D.M.)

THE membrane theory is described from first principles and the methods of calculation are derived for shells of cylindrical, conical and spherical shapes. Although the book, which is in the German language, deals exhaustively with the mathematics of this type of shell structure, only a small portion of the analysis appears to have practical significance. This disadvantage may be overcome in the second volume. A graphical method is described which is suitable for any shell the shape of which is a surface of revolution subjected to vertical, horizontal and wind loading. Chapters are devoted to deformations arising from membrane action, bending of shells which involves hyperbolic differential equations, the effect upon a circular wall of a non-uniform rise of temperature, and the design of ring beams in structures composed of spherical, cylindrical or conical elements. Six numerical examples are given of cylindrical and conical tanks.

"Stahlbeton Tabellen." By K. Hofacker. (Zurich: Verlag Leemann. 1960. Price 15 Swiss francs.)

ABOUT sixty tables are given which, with the numerous explanatory examples, should be of considerable use to designers of reinforced concrete rectangular beams and tee-beams when calculating in the metric system, although many of the factors are non-dimensional and are therefore of wider application. Some of the tables are applicable to columns subjected to bending.

Books Received.

"The Design of Modern Steel Structures." By L. E. Grinter. (New York: The Macmillan Co. 1960. Second edition. 500 pages.)

This book deals with the design of steelwork in accordance with the latest practice in the U.S.A.

"Surveying." By A. H. P. Gillett. (London: The Sunday Times. 1960. Price 3*s*. 6*d*.)

A paper-back in a series of career books and dealing in a popular manner with the work of various types of surveyors.

"Bestimmungen des Deutschen Ausschusses für Stahlbeton." (Berlin: Wilhelm Ernst & Sohn. 1960. Seventh edition. Price 19.80 D.M.)

A compilation of upwards of thirty German regulations applying to reinforced concrete.

"Mobile Tower Cranes for Two and Three-storey Buildings." (London: H.M.S.O. 1960. Price 4*s*. 6*d*.)

Illustrated account of uses of rail-mounted and crawler-track cranes with notes on costs and site experience.

Tests of Joints Subjected to Bending and Shearing in Precast Members.

By W. J. LARNACH, M.Sc., A.M.I.C.E.

JOINTS in reinforced concrete members are usually at or near the planes of contraflexure to avoid large bending moments acting at the joints. However, it would be convenient to have a wider choice in the possible position of joints, and for this a knowledge of their resistance to bending is required. Mr. C. H. Dobbie and Mr. R. L. Wajda [1] have described the behaviour of one type of joint subjected to bending. The present article describes the behaviour of beams with several types of joint, comparing them with equivalent beams without joints, to assess the efficiency of the joints in transmitting bending moments.

The cross-section of all members tested was 6 in. by 6 in. A shallow beam was chosen since it was felt that the conditions are more severe for such members. A mould 10 ft. long enabled a non-jointed beam about 5 ft. long and the two components of a beam with joints, and of the same total length, to be cast simultaneously.

The specimens were cast on their sides, and were of 1 : 7 concrete with aggregate of ¾ in. maximum size and a water-cement ratio of 0·64. Compaction was by vibration. At the same time 6-in. cubes were made for compressive-strength tests; a cube-strength between 4000 and 5000 lb. per square inch at twenty-eight days was aimed at. All concrete was cured under water at 14 to 18 deg. C.

Details of the Joints.

The joints tested and the type of loading employed are shown diagrammatically in *Fig.* 1, and *Fig.* 2 gives details of the reinforcement and connections.

No.	TYPE OF JOINT	LOADING	Cube Strength	Jointed Beam Failure Moment Estimated	Jointed Beam Failure Moment Actual	Plain Beam Failure Moment	Joint Efficiency
1	Square Scarf: 1 bolt each side	Beam & Joint 1'-0" 1'-0" 4'-6"	5470 lb. per sq. in.	17·6 in.-ton.	14·8 in.-ton.	43·5 in.-ton.	34 per cent.
2	Sloping Scarf: 1 bolt each side		3830	15·9	18·0	49·8	36
3	Dowelled		4650		32·6	36·4	88
4	Covered Butt: 5 bolts each side		5030		60·3	64·8	96
5	Sloping scarf: 3 bolts total.		4740	25·4	30·0	62·3	48
6	Ditto	1'-1½" 5'-0"	5430	28·9	37·0	75·0	50

Fig. 1.—Types of Joint and Results of Tests.

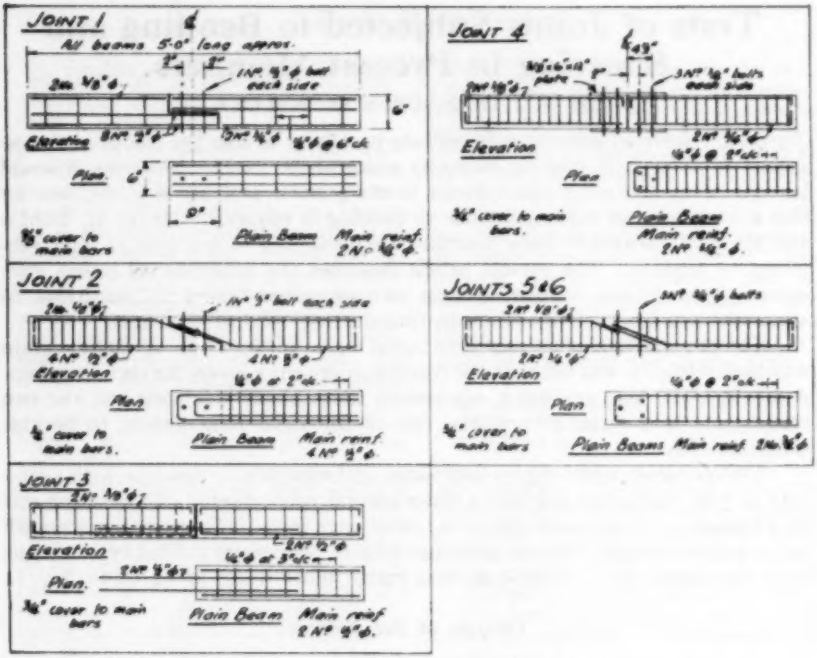


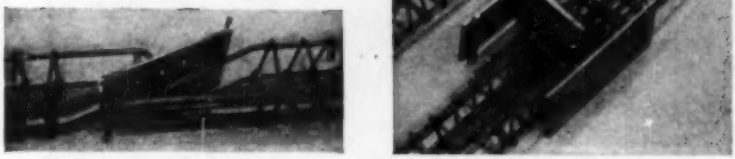
Fig. 2.—Reinforcement Details.

Holes for connecting bolts were formed by placing bars of appropriate diameter in $\frac{1}{4}$-in. plates separating the components of the beams. The bars were in thick rubber sleeves which were extracted when the moulds were stripped, enabling the bars to be easily removed, and leaving holes in the components correctly registered. *Figs.* 3(a) and (b) show the details of the separating plates and reinforcement for joints 2 and 4.

The dowels for joint 3 were cast integrally with one component and, passing through a $\frac{1}{4}$-in. separating plate, projected into a cavity formed in the other component. To avoid premature failure due to shearing, all specimens were provided with $\frac{1}{4}$-in. diameter stirrups.

Assembly of Beams with Joints.

After curing, each pair of components was clamped together in proper alignment whilst the space formerly occupied by the separating plate was filled with a mixture of 1 : 4 high-alumina cement and sand, having a water-cement ratio of 0·4. Where necessary, mild-steel bearing pads, $\frac{1}{8}$ in. thick, for the connecting bolts were embedded in a similar mortar. After the mortar had gained sufficient strength the joint was completed by inserting and tightening the bolts. Joint 3 required the use of grout, which was injected under pressure.



(a) Joint Type 2. (b) Joint Type 4.

Fig. 3.

Testing.

For each type of joint, a beam with a joint and a corresponding beam without a joint, were tested. The beams were freely supported at a span of 4 ft. 6 in. Equal concentrated loads were applied at 2 ft. centres as shown in *Fig.* 1. The joints were central in the zone of constant bending moment. Conditions for joint 6 were changed, to introduce high shearing forces at the joint. Electric resistance strain gauges were affixed to all connecting bolts. All deflections were measured at sections corresponding to the centre lines of the joints.

Results.

Since the strength of the concrete of each pair of specimens was not constant, the "joint efficiency", which is the bending strength of a jointed beam divided by the bending strength of a plain beam, has been determined in each case for purposes of comparison. Cube-strengths and joint efficiencies are shown in *Fig.* 1.

Joint 1.—Failure was expected either at the section weakened by the presence of the bolt hole, or at the sharp corner where the cross-section changed abruptly from half the depth of the member to the full depth. In fact, flexural failure occurred in the latter position, with simultaneous bond-failure of the poorly anchored reinforcement at the joint.

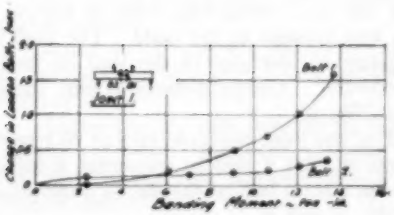


Fig. 4.

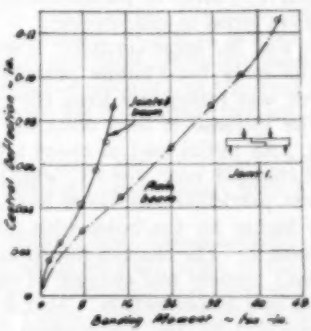


Fig. 5.

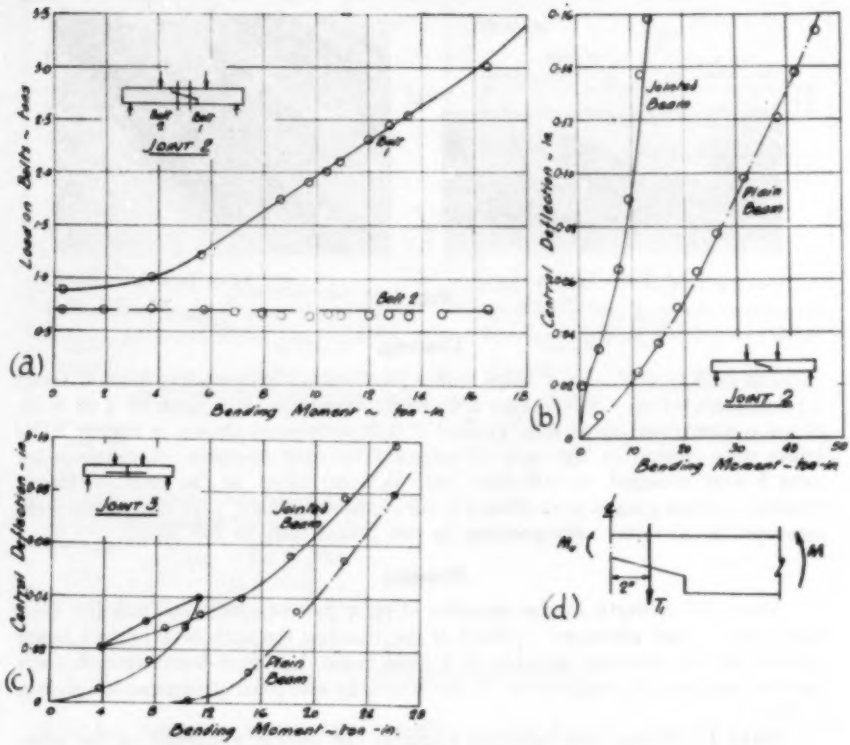


Fig. 6.

The part of the beam which is about 3 in. deep at the joint can be treated as a "steel beam", for which the bending moment causing failure $M_u = At_y a$, in which A = areas of reinforcement in tension and compression = 0·785 sq. in., t_y = yield stress in reinforcement = 18·0 tons per square inch by test, a = lever arm = 1·25 in. approximately; whence $M_u = 17{\cdot}7$ in.-tons.

The early bond-failure probably explains why the experimental ultimate moment was rather less than the calculated figures.

The increase of load occurring on the ½-in. connecting bolts is shown in *Fig.* 4. An initial load of about 1 ton was induced in the bolts. The load on bolt 2 changed very slightly, whilst significant increases in the load on bolt 1 occurred, particularly as failure was approached. Central deflections were significantly higher for the beam with joints (*Fig.* 5).

Joint 2.—The changes incorporated in this joint were an attempt to remove the sharp corners and changes of section of joint 1 and to encourage a failure to occur at a deeper section. Also, to avoid a bond-failure, satisfactory anchorage was provided for the reinforcement (*Figs.* 2 and 3).

Failure again occurred where the effective section was about 3 in. deep,

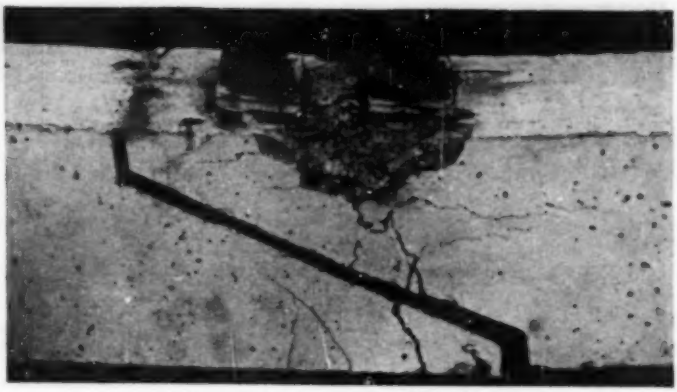


Fig. 7.—Joint Type 5 after Failure.

that is at the centre line of the joint. The simplified actions at the joint are shown in *Fig.* 6(d). The ultimate moment of the over-reinforced section at failure is given by

$$M_u = \tfrac{1}{3}u_c bd^2 = \tfrac{1}{3}(0{\cdot}85 \times 3830) \times 6 \times 1{\cdot}88^2 = 10{\cdot}3 \text{ in.-tons},$$

in which u_c is the cube-strength.

Load T_1 on bolt 1 at failure = 3·48 tons,

$$M = M_u + (T_1 \times 2) = 17{\cdot}26 \text{ in.-tons}.$$

This compares well with the experimentally-determined moment causing failure of 18·0 in.-tons. The load on bolt 2 is sensibly unchanged during the test, whilst the tension in bolt 1 increases to a value almost equal to the yield load of 3·53 tons [*Fig.* 6(a)]. Also, the stiffness of the beam with joints is considerably less than that of the plain beam [*Fig.* 6(b)].

Joint 3.—Failure was initiated due to the failure of bond of the dowel bars in the grouted half of the joint.

The load-deflection curve [*Fig.* 6(d)] shows a high residual deflection on unloading from about 11 in.-tons. It is possible that this is an indication of slipping of the bars at low bending moments; nevertheless, the deflection curve on re-loading above 11 in.-tons appears to conform well with the initial part of the curve. The deflections of the jointed and plain beams are more nearly equal than those for joints 1 and 2. Though the efficiency of the joint is high, it is considered that it might be improved by the use of deformed bars and grout of higher quality.

Joint 4.—The plates were adequate to develop the full estimated moment of resistance of the plain beam, without causing bearing failure with ¾-in. connecting bolts.

Under test the joint was 100 per cent. efficient, since failure actually occurred outside the joint. However, cracking did occur on both sides of the joint at the weakest concrete sections, that is, those containing two bolt holes. The central deflections of the beam at the joint were about 1·5 times those of the plain beam.

Fig. 8.—Joint Type 6 after Failure.

Joint 5.—In an attempt to increase the efficiency of joint 2, the number of connecting bolts was increased to three, and their diameter to $\frac{3}{4}$ in.; an initial load of about 1·75 tons was induced in each bolt. The area of main reinforcement was slightly greater than that for joint 2. The load on bolt 2 was again little changed, whilst the load on bolts 1 increased steadily with increasing applied moment. Deflections at the joint were 2·4 times those of the plain beam at half the moment causing failure, and 5·8 times at failure. The joint failed in line with bolts 1 (*Fig.* 7), where the calculated ultimate moment was 25·4 in.-tons; the actual moment causing failure was 30 in.-tons.

Joint 6.—The details of the jointed and plain beams were exactly as for joint 5. The loading conditions were amended to introduce a high shearing force at the joint (3·9 tons at failure). The failure is illustrated in *Fig.* 8, which shows that it is confined to the upper part of the joint.

The behaviour of the joint was similar to that of joint 5 with respect to deflections and change of load on the bolts.

Conclusions.

The following conclusions were reached for the series of beams tested. The introduction of a joint materially reduces the stiffness of a beam. Joints 3 and 4 affected the stiffness least. In bolted joints a significant change in load appears to occur only on those bolts (bolts 1) situated in the "top half" of the joint. The loads in bolts 1 can approach the yield values. If large bending moments are to be transmitted with this type of joint it would clearly be wise to use high-tensile bolts, which would permit the use of large initial clamping forces, whilst avoiding the possibility of premature failure of the joint due to early yielding of the bolts; also the effective area of concrete would be increased due to the use of smaller bolts.

The efficiencies of the square and sloping scarf joints, 1 and 2, with two bolts were practically the same as that found by Mr. C. H. Dobbie and Mr. R. L. Wajda. (It should be noted, however, that the efficiency they determined was

based on an estimated plain-beam strength and not on an experimentally determined figure.) The use of more and larger bolts (joints 5 and 6) can increase the efficiency to about 50 per cent., which would seem to be about the limit for this form of joint with mild steel bolts.

The presence of a considerable shearing force (joint 6) did not appear to affect the efficiency of the sloping scarf joint. It is suggested, however, that additional reinforcement against shearing would be desirable in the region of joints under the action of shearing forces.

Extremely high efficiencies can be achieved with dowelled and butt joints (joints 3 and 4). These joints would probably be more difficult than the others to make on site, but may be economical if the transmission of higher bending moments were necessary.

The efficiency of all joints, except the dowelled joint, may be expected to be rather less than those in these tests if, for the sake of appearance, the cover plates and bolt bearing plates were to be embedded in the beams.

The strength of the sloping scarf joints with three bolts can be estimated using the formula $M_u = \frac{1}{3} u_c b d^2$ for the over-reinforced joints at failure.

The beams were manufactured and tested in the Department of Civil Engineering of the University of Bristol.

1 —C. H. Dobbie and R. L. Wajda. "Joints in Precast Frames." Concrete and Constructional Engineering, April 1956.

The Use of Strand for Prestressed Concrete.

At a recent meeting of the Prestressed Concrete Development Group at Cardiff, Mr. E. W. H. Gifford made the following comments on the use of strand for prestressing concrete.

In post-tensioning, stranded cable has the advantage that it allows a greater concentration of force than parallel-wire cables. This can lead to the design of slimmer concrete sections and to greater eccentricities of prestressing force, thus reducing the total force required. The labour in assembling and tensioning the cable can also be reduced. Another advantage is that machined steel saddles can be used and considerably reduce losses due to friction. A disadvantage is that the cost, per ton of available prestressing force, is greater for strand than for wire and generally the prestressing equipment is heavier. Strand is most suitable for heavily loaded beams of long span.

Anchorages should prevent the inner wires from slipping relative to the outer ones and should not impart such stresses to the wires that some are broken by a combination of shearing and tension. Moreover, the fixing of the anchorage to the strand should be a simple operation and the split-wedge type of grip with a steel barrel appears to be most suitable. By using machined steel saddles at the places where cables change direction losses due to friction can be as low as 5 per cent. even with long cables. Strand is normally delivered in 3300-lb. reels and is generally easier to handle than the equivalent parallel-wire cable.

Pretensioned strand may be used economically in small precast units if the units have been designed specifically for stranded cables and if the prestressing bed is suitable.

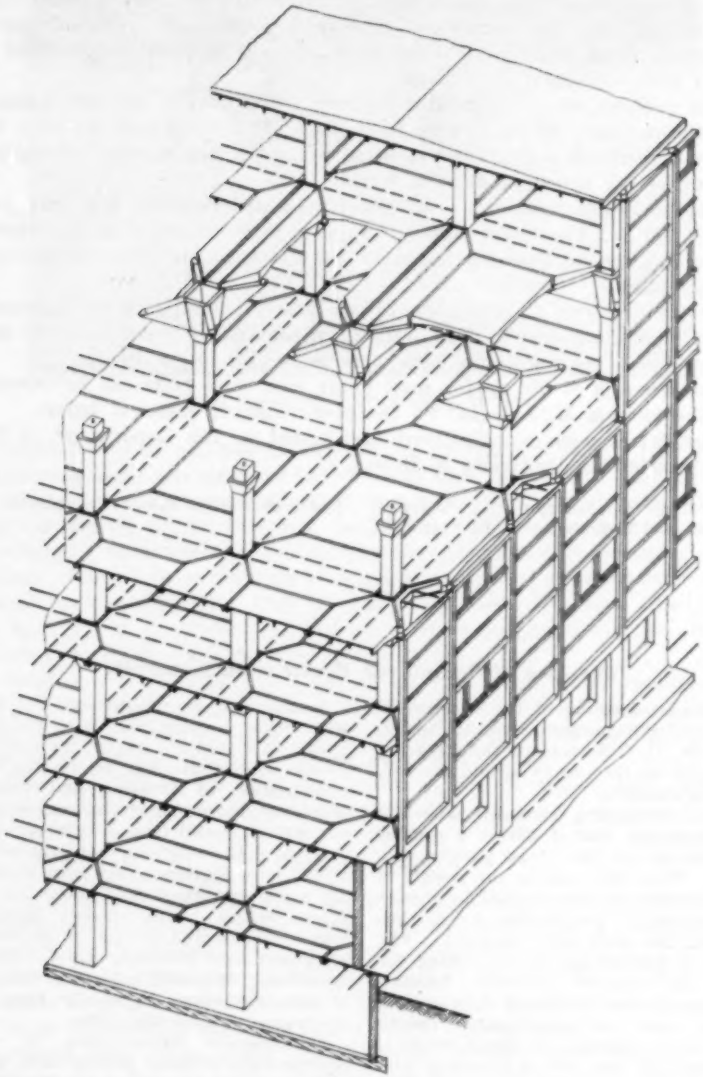


Fig. 1.

(*See facing page.*)

Prestressed Precast Concrete Multiple-story Warehouse.

A SYSTEM used in Poland for the construction of warehouses and factories comprising precast columns and slabs, was described in this journal for November 1959. Prestressing has been applied to this standard system in the construction of a warehouse at Włocławeck. The main precast members and their assembly are described in the previous article and are shown in *Fig.* 1. In the warehouse at Włocławeck the floors are prestressed in two directions at each story to provide continuity.

dead and imposed loads, so that under working conditions each floor acts as a series of cantilevers from the columns.

Tests.

A test structure was erected near by and loaded with aggregate to 1·8 times the imposed load of 1500 kg. per square metre (307 lb. per square foot) for which the structure was designed. Cracks which formed in the ribs of the square floor member and a deflection of 0·3 cm. (0·12 in.) at the centre of the panel were

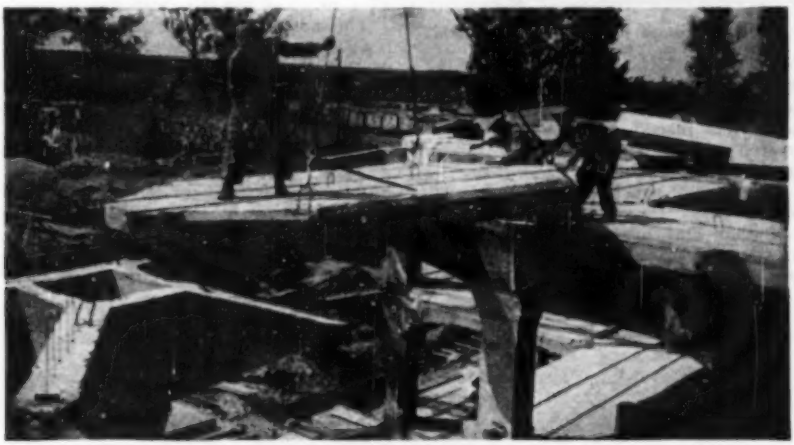


Fig. 2.—Erecting Precast Members.

Prestressing.

Two grooves were cast in the upper slab of the polygonal floor members so that, when the members were in place, prestressing cables could be laid in the grooves and extend completely across the building close to the faces of the columns (*Fig.* 1). The prestressing cables each comprise twelve 5-mm. (0·2-in.) wires and were prestressed by a method similar to the Freyssinet method and anchored against square steel plates embedded in U-shape members (*Fig.* 4) at the edges of the building. The tension in the cables is such that there is no positive bending moment at mid-span under the action of the only signs of overloading. Tests on this structure indicated that, due to the shape of the ribs and the precast column-head, some load was transmitted to the column by arch action.

Construction.

The columns are at intervals of 6 m. (about 19 ft. 8 in.) in both directions. The column-heads were first set on the columns and the polygonal floor members placed in position (*Fig.* 2). The square floor members were then placed in position and the joints between all the precast members were concreted. The prestressing cables were then laid in

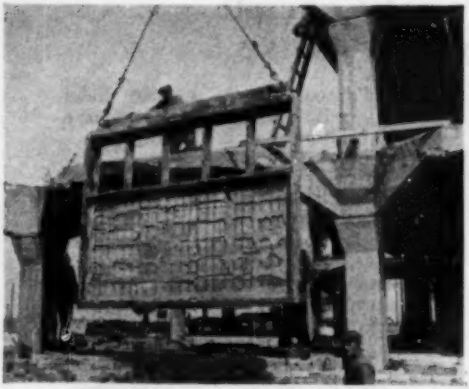


Fig. 3.—Erection of Precast Wall Panels.

grooves in the slab and were tensioned. Reinforcement of 4·5 mm. ($\frac{1}{6}$ in.) diameter was placed over the cables and covered with a layer of cement mortar embedding the cables.

Precast wall panels (*Fig.* 3) and brickwork formed the external walls. The stair-wells and lift-shafts pass through the floors in place of some of the square floor members. All the precast members

Fig. 4.—Cable Anchors in Precast Members.

are about the same weight and were set in place by means of a tower crane of 45 t.-m. (145 ton-ft.) capacity.

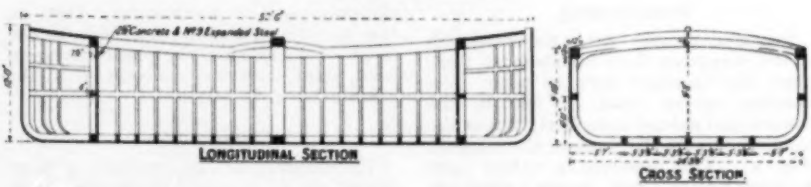
The materials used per square metre of floor area are as follows: precast concrete, 0·1 cu. m.; prestressing wire, 1·2 kg.; insitu concrete, 0·03 cu. m.; reinforcement, 11 kg. The materials used for each square yard of floor are: precast concrete, 0·11 cu. yd.; prestressing wire, 2·21 lb.; cast-insitu concrete, 0·033 cu. yd.; and reinforcement, 20·25 lb.

J. Draguła, W. Panorski, and W. Zalewski collaborated in the design with the Standard Designs Bureau for Industrial Buildings of Poland.

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering", December, 1910.

A Concrete Barge.



"The barge illustrated was constructed by the licensees of the Expanded Metal Co. in Italy. It was designed for towing. The concrete was composed of 3 parts ¾-in. aggregate, 1½ parts of sand and 1 part of Portland cement. It was built in 1906, since when it has been in constant use for coal traffic, carrying about 150 tons, and is still * giving satisfactory results."

* In 1910.

Design of Biaxially-loaded Columns by the Ultimate-load Method.—III.*

By F. N. PANNELL, M.Sc., B.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E.

THE appendices in which are set out the equations of static equilibrium for multiple-bar columns, eight-bar columns, and four-bar columns, and to which references were made in Parts I and II of this article, are as follows.

APPENDIX I.

EQUATIONS OF STATIC EQUILIBRIUM FOR MULTIPLE-BAR COLUMNS.

The following values are assumed; ultimate concrete strain $\epsilon_u = 0{\cdot}33$ per cent. Modulus of elasticity of mild-steel reinforcement $E_s = 30 \times 10^6$ lb. per square inch, $X = 0{\cdot}85K$.

From *Fig.* 13; $X_1 = \dfrac{Kp_{sc}}{0{\cdot}5\epsilon_u E_s}$, $X_2 = \dfrac{Kp_{st}}{0{\cdot}5\epsilon_u E_s}$, the elastic steel stress $t = \dfrac{0{\cdot}5\epsilon_u E_s(1-K)}{K}$, between the limits p_{st} and $-p_{sc}$.

Let $A_1 = \dfrac{rd_1}{2p_{sc}(d-2d_2)}$.

$$A_2 = p_{sc}\left(K - \frac{d_2}{d_1} - \frac{X_1}{2}\right) - p_{st}\left(1 - K - \frac{X_2}{2}\right).$$

$$A_3 = p_{sc}\left(K - \frac{d_2}{d_1} - \frac{X_1}{2}\right) - \frac{t}{2}(1-K).$$

$$A_4 = p_{sc}\left(K - \frac{d_2}{d_1} - X_1\right)(1 - K + X_1)\frac{d_1}{2d}.$$

$$A_5 = \frac{p_{sc}X_1}{2}\left[0{\cdot}5 + \frac{d_1}{d}\left(\frac{2X_1}{3} - K\right)\right].$$

$$A_6 = p_{st}(1 - K - X_2)\left[\frac{d_1}{2d}(K + X_2) - \frac{d_2}{2d}\right].$$

$$A_7 = \frac{p_{st}X_2}{2}\left[\frac{d_1}{d}\left(K + \frac{2X_2}{3}\right) - 0{\cdot}5\right].$$

$$A_8 = \frac{t}{2}(1-K)\left[\frac{d_1}{3d}(2 + K) - 0{\cdot}5\right].$$

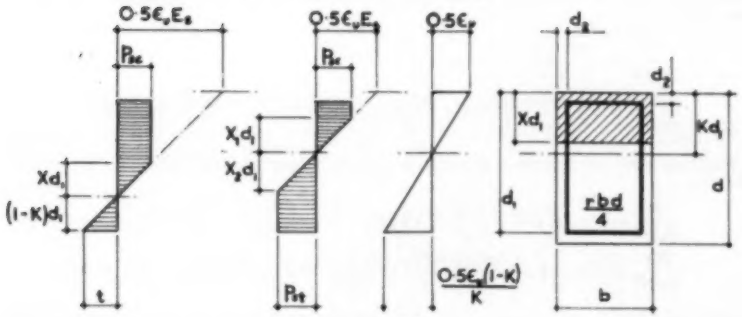


Fig. 13.

(a).—Stress in Reinforcement at Failure due to Compression. (b).—Stress in Reinforcement at Failure due to Tension. (c).—Strain of Concrete. (d).—Cross-section.

* Continued from November, 1960.

Rectangular Columns Subjected to Bending about a Major Axis (*Fig.* 13).

FAILURE DUE TO TENSION.—

$$\frac{P}{p_{cc}bd} = \frac{Xd_1}{d} + \frac{r}{4p_{cc}}(p_{sc} - p_{st}) + A_1A_2.$$

$$\frac{M_g}{p_{cc}bd^2} = \frac{Xd_1}{2d}\left(1 - \frac{Xd_1}{d}\right) + \frac{r}{4p_{cc}}(p_{sc} + p_{st})\left(0{\cdot}5 - \frac{d_2}{d}\right) + A_1(A_4 + A_5 + A_6 + A_7).$$

FAILURE DUE TO COMPRESSION.—

$$\frac{P}{p_{cc}bd} = \frac{Xd_1}{d} + \frac{r}{4p_{cc}}(p_{sc} - t) + A_1A_2.$$

$$\frac{M_g}{p_{cc}bd^2} = \frac{Xd_1}{2d}\left(1 - \frac{Xd_1}{d}\right) + \frac{r}{4p_{cc}}(p_{sc} + t)\left(0{\cdot}5 - \frac{d_2}{d}\right) + A_1(A_4 + A_5 + A_8)$$

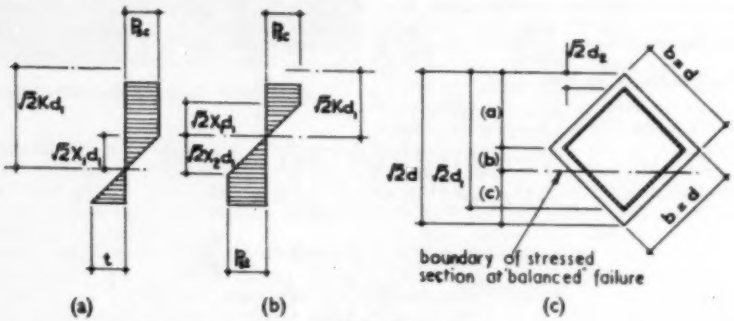


Fig. 14.

(a).—Stress in Reinforcement at Failure due to Compression. (b).—Stress in Reinforcement at Failure due to Tension. (c).—Cross-section.

Square Columns Subjected to Bending about a Diagonal (*Fig.* 14).

FAILURE DUE TO TENSION WHEN THE LOWER LIMIT OF THE STRESSED SECTION IS IN ZONE (a).—

$$\frac{P}{p_{cc}bd} = 2\left(\frac{Xd_1}{d}\right)^2 + 2A_1A_2.$$

$$\frac{M_d}{p_{cc}bd^2} = 2\sqrt{2}\left(\frac{Xd_1}{d}\right)^2\left(0{\cdot}5 - \frac{2Xd_1}{3d}\right) + 2\sqrt{2}A_1(A_4 + A_5 + A_6 + A_7).$$

FAILURE DUE TO TENSION WHEN THE LOWER LIMIT OF THE STRESSED SECTION IS IN ZONE (b).—

$$\frac{P}{p_{cc}bd} = 1 - 2\left(1 - \frac{Xd_1}{d}\right)^2 + 2A_1A_2.$$

$$\frac{M_d}{p_{cc}bd^2} = 2\sqrt{2}\left(1 - \frac{Xd_1}{d}\right)^2\left(\frac{2Xd_1}{3} - \frac{1}{6}\right) + 2\sqrt{2}A_1(A_4 + A_5 + A_6 + A_7).$$

FAILURE DUE TO COMPRESSION WHEN THE LOWER LIMIT OF THE STRESSED SECTION IS IN ZONE (c).—

$$\frac{P}{p_{cc}bd} = 1 - 2\left(1 - \frac{Xd_1}{d}\right)^2 + 2A_1A_2.$$

$$\frac{M_d}{p_{cc}bd^2} = 2\sqrt{2}\left(1 - \frac{Xd_1}{d}\right)^2\left(\frac{2Xd_1}{3} - \frac{1}{6}\right) + 2\sqrt{2}A_1(A_4 + A_5 + A_8).$$

FAILURE DUE TO AXIAL LOAD.—

$$\frac{P}{p_{cc}bd} = 1 + r\frac{p_{sc}}{p_{cc}}.$$

APPENDIX II.

EQUATIONS OF STATIC EQUILIBRIUM FOR EIGHT-BAR COLUMNS.

The following values are assumed; $\epsilon_u = 0{\cdot}33$ per cent. $E_s = 30 \times 10^6$ lb. per square inch $t = \dfrac{\text{elastic steel stress}}{p_{su}} = \dfrac{0{\cdot}425\epsilon_u E_s}{p_{sc}}\left(\dfrac{Q}{R} - 1{\cdot}175\right)$ between the limits $+1{\cdot}111$ and $-1{\cdot}0$ and where Q and R are given for each case.

Rectangular Columns Subjected to Bending about a Major Axis [*Fig.* 15 (a)]. $R = Xd$; for t_1, $Q_1 = 0{\cdot}5d$; for t_2, $Q_2 = d - d_2$; $t_0 = -1{\cdot}0$.

$$\frac{P}{p_{cc}bd} = X - r\frac{p_{sc}}{8p_{cc}}(3t_0 + 2t_1 + 3t_2).$$

$$\frac{M_g}{p_{cc}bd^2} = \frac{X}{2}(1 - X) + r\frac{p_{sc}}{8p_{cc}}(3t_2 - 3t_0)\left(0{\cdot}5 - \frac{d_2}{d}\right).$$

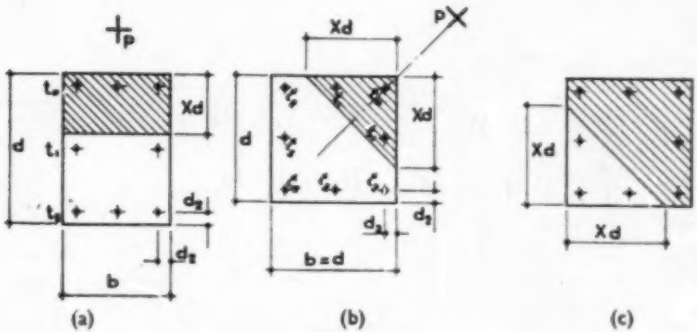


Fig. 15.

Square Column Subjected to Bending about a Diagonal.

STRESSED SECTION LESS THAN HALF COLUMN AREA [*Fig.* 15(b)].—$R = Xd$; for t_1, $Q_1 = d_2 + 0{\cdot}5d$; for t_2, $Q_2 = d$; for t_3, $Q_3 = 1{\cdot}5d - d_2$; for t_4, $Q_4 = 2(d - d_2)$; and $t_0 = -1{\cdot}0$.

$$\frac{P}{p_{cc}bd} = \frac{X^2}{2} - r\frac{p_{sc}}{8p_{cc}}(t_0 + 2t_1 + 2t_2 + 2t_3 + t_4).$$

$$\frac{Md}{p_{cc}bd^3} = \frac{X^2}{\sqrt{2}}\left(\frac{1}{2} - \frac{X}{3}\right) + \frac{\sqrt{2}\,rp_{sc}}{8p_{cc}}(t_3 + t_4 - t_0 - t_1)\left(\frac{1}{2} - \frac{d_2}{d}\right).$$

STRESSED SECTION MORE THAN HALF COLUMN AREA [*Fig.* 15(c)].—$R = d(2 - X)$. Values of Q are as for the previous case.

$$\frac{P}{p_{cc}bd} = 1 - \frac{X^2}{2} - r\frac{p_{sc}}{8p_{cc}}(t_0 + 2t_1 + 2t_2 + 2t_3 + t_4)$$

$\dfrac{M}{p_{cc}bd^2}$ is as for the previous case.

APPENDIX III.

EQUATIONS OF STATIC EQUILIBRIUM FOR BIAXIALLY LOADED FOUR-BAR COLUMNS.

The following values are assumed; the ultimate concrete strain $\epsilon_u = 0{\cdot}33$ per cent, $E_s = 30 \times 10^6$ lb. per square inch, $t = \dfrac{\text{elastic steel stress}}{p_{sc}} = \dfrac{0{\cdot}425\epsilon_u E_s}{p_{sc}}\left(\dfrac{Q}{R} - 1{\cdot}175\right)$ between the limits $+1{\cdot}111$ and $-1{\cdot}0$.

For t_1, $Q_1 = d_2 + (b - d_2)\tan\alpha$; for t_2, $Q_2 = (d - d_2) + (b - d_2)\tan\alpha$; for t_3, $Q_3 = (d - d_2) + d_2\tan\alpha$; and $t_0 = -1{\cdot}0$.

$\tan\alpha$ and R are given for each case.

Case I [*Fig.* 16(a)].—$R = X_2 d$, $\tan\alpha = (X_2 - X_1)\dfrac{d}{b}$.

$$\frac{P}{p_{cc}bd} = \frac{X_1 + X_2}{2} - r\frac{p_{sc}}{4p_{cc}}(t_0 + t_1 + t_2 + t_3).$$

$$\frac{Px}{p_{cc}bd^2} = \frac{X_2 - X_1}{12} + r\frac{p_{sc}}{4p_{cc}}(t_1 + t_2 - t_3 - t_0)\left(\frac{1}{2} - \frac{d_2}{b}\right).$$

$$\frac{Py}{p_{cc}bd^2} = \frac{X_1}{2}(1 - X_1) + \frac{X_2 - X_1}{2}\left(0{\cdot}5 - X_1 - \frac{X_2 - X_1}{3}\right) + r\frac{p_{sc}}{4p_{cc}}(t_2 + t_3 - t_0 - t_1)\left(\frac{1}{2} - \frac{d_2}{d}\right).$$

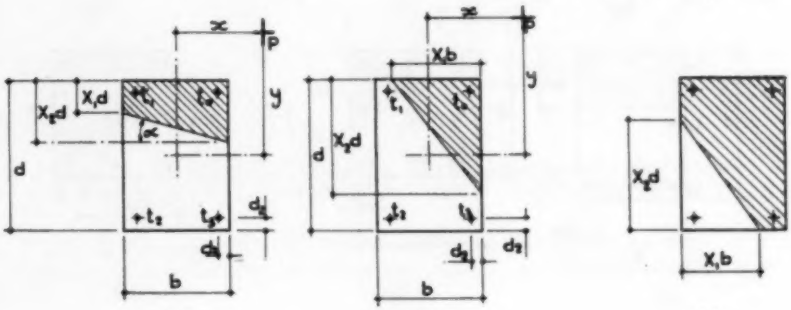


Fig. 16.

Case II [*Fig.* 16(b)].—$R = X_2 d$, $\tan\alpha = \dfrac{X_2 d}{X_1 b}$.

$$\frac{P}{p_{cc}bd} = \frac{X_1 X_2}{2} - r\frac{p_{sc}}{4p_{cc}}(t_0 + t_1 + t_2 + t_3).$$

$$\frac{Px}{p_{cc}bd^2} = \frac{X_1 X_2}{2}\left(\frac{1}{2} - \frac{X_1}{3}\right) + r\frac{p_{sc}}{4p_{cc}}(t_1 + t_2 - t_0 - t_3)\left(\frac{1}{2} - \frac{d_2}{b}\right)$$

$$\frac{Py}{p_{cc}bd^2} = \frac{X_1 X_2}{2}\left(\frac{1}{2} - \frac{X_2}{3}\right) + r\frac{p_{sc}}{4p_{cc}}(t_2 + t_3 - t_0 - t_1)\left(\frac{1}{2} - \frac{d_2}{d}\right).$$

Case III [*Fig.* 16(c)].—$R = d(1 - X_2) + b\tan\alpha$, $\tan\alpha = \dfrac{X_2 d}{X_1 b}$.

$$\frac{P}{p_{cc}bd} = 1 - \frac{X_1 X_2}{2} - r\frac{p_{sc}}{4p_{cc}}(t_0 + t_1 + t_2 + t_3).$$

$\dfrac{Px}{p_{cc}bd^2}$ and $\dfrac{Py}{p_{cc}bd^2}$ are as for Case II.

Civil Engineering at Hinkley Point Nuclear Power Station.

THE civil engineering work at the nuclear power station at Hinkley Point on the Bristol Channel, near Bridgwater, is now almost completed. This station, which is the fourth of the nuclear power stations for the supply of electricity for civil purposes to be installed in Great Britain, will have a capacity of 500 megawatts. The type and arrangement of the structures are in general similar to the other stations * but some of the civil engineering works differ.

A general view of the site last year

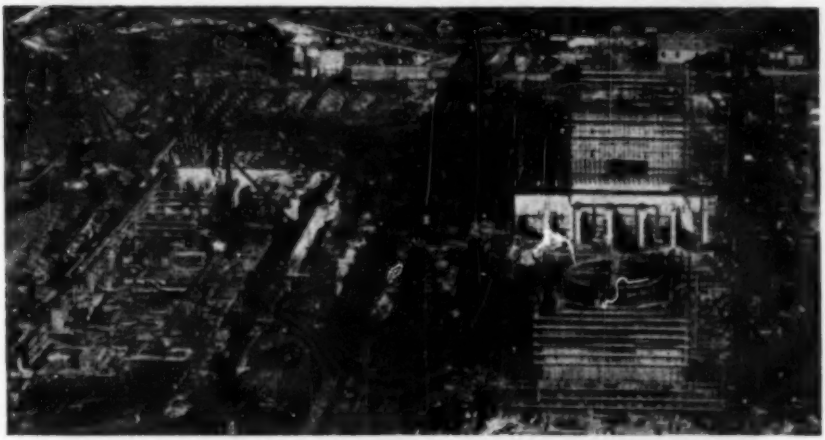


Fig. 1.

is shown in *Fig.* 1. The two reactor structures are to the right. The turbine hall is to the left of the reactors. The intake and outfall works for the cooling-water system are in the estuary beyond the left-hand side of the illustration. A feature of the intake works is a caisson which was constructed on the shore and floated into position. The extensive temporary works include a camp to accommodate about a thousand men, a 400-ton goliath crane, and a wharf about five miles from the site. The goliath crane, which was erected in July 1959 and was not primarily intended for use in connection with the civil engineering works, has a span of 250 ft. and a clear height of 200 ft.; the foundations of the two single-rail tracks contain about 2500 cu. yd. of concrete. The wharf, which is used for unloading heavy equipment brought by sea, can accommodate vessels up to 1500 tons displacement. Radio-telephones are still being used to communicate with vessels approaching the wharf.

Part of the site of the station is reclaimed land, the seaward boundary of which is formed by a wall of plain concrete 3500 ft. long and having a maximum height of 21 ft.; the wall contains about 19,000 cu. yd. of concrete.

Reactor Structures.

The reactor structures comprise primarily inner and outer biological shielding

* A preliminary description of the station at Hinkley Point is given in this journal for July 1958. The civil engineering work at the station at Berkeley, Gloucestershire, is described in this journal for June 1959, and that at Hunterston, Scotland, for February 1960. Some particulars of the concrete at the station at Bradwell, Essex, are given in the number for May 1960. The construction of a fifth station was commenced at Trawsfynydd, North Wales, in the summer of 1959.

Fig. 2.—Reactor No. 1 showing Cranes.

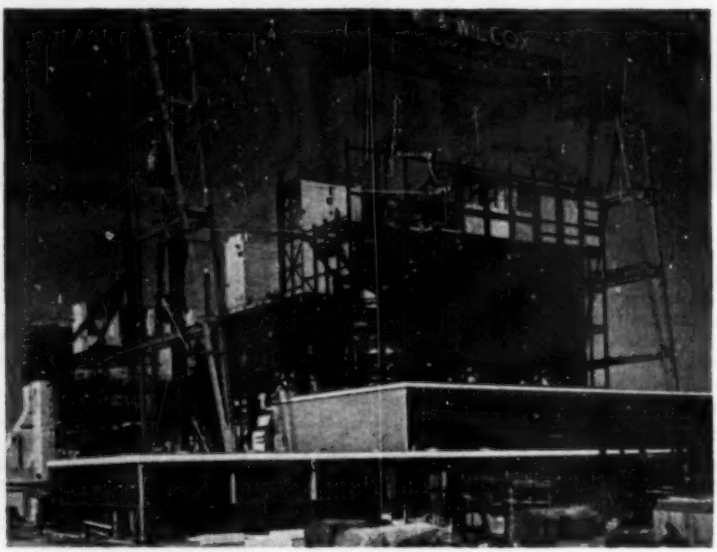


Fig. 3.

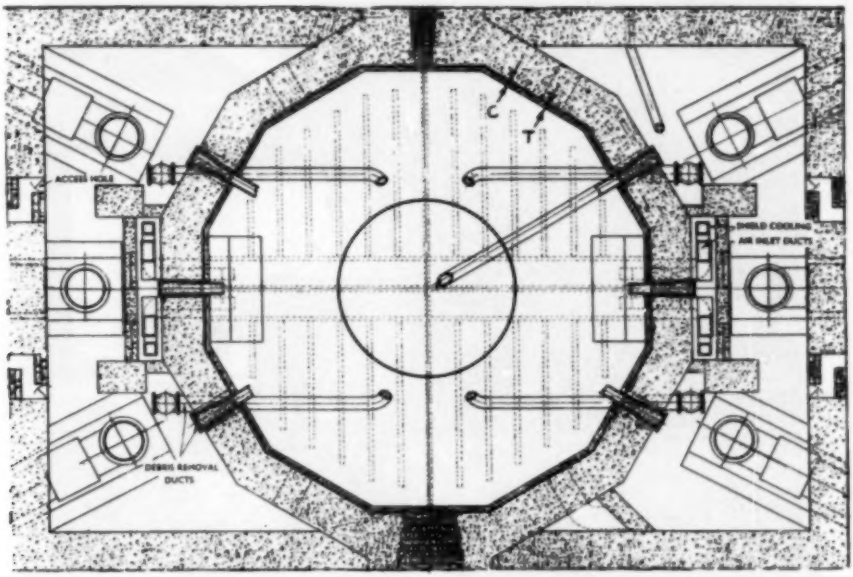


Fig. 4.—Sectional Plan of Reactor showing the Biological and Thermal Shields.

walls (*Fig.* 4) and a top shielding slab. The inner walls are 100 ft. high and 7 ft. thick and form a polygon which has twelve sides and is 76 ft. across the flats. The walls are lined with thermal shielding panels (T), the manufacture of which is described later. The secondary shielding walls are 6 ft. thick and enclose a space about 100 ft. wide and 140 ft. long surrounding the inner walls. The shielding slab over the reactor will be 11 ft. thick. The ground under the reactor structures is shale and limestone. More than 60,000 cu. yd. of rock have been excavated for the main rafts and for the deeper foundations of the adjacent gas-blower houses. The pressure on the ground under the reactors is about 10 tons per square foot.

The reactor structures are shown in course of construction in *Figs.* 1, 2 and 3. In the latter illustration, the buildings in the foreground are the carbon-dioxide store and the blower house for Reactor No. 1. The three steam-raising units on the eastern side of Reactor No. 1 are seen as are also the goliath crane and the concrete shielding behind the structural-steel framing.

The concrete in the shielding walls and roof has an average density of not less than 140 lb. per cubic foot when completely dry, and a crushing strength of not less than 3500 lb. per square inch at 28 days. It is placed in blocks, separated by gaps, in a sequence such that no concrete is placed against other concrete until the latter concrete is at least three days old. By this means, stresses due to shrinking and thermal changes are reduced.

The shuttering comprises panels 5 ft. by 8 ft. 6 in. formed of 1½-in. boards attached to steel soldiers. The inner face of the inner shielding wall is required to be accurate to within plus ½ in. and minus ½ in. and the wall must be not less than the specified thickness. The positions of small metal inserts for the attachment of mechanical equipment are required to be correct within ½ in.

The concrete in the top shielding slab, which is 11 ft. 6 in. thick and is perforated with numerous holes through which the

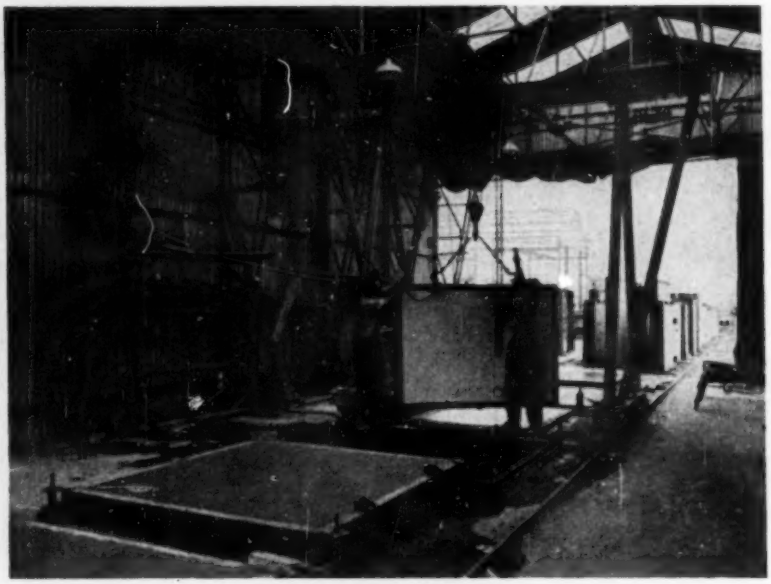


Fig. 5.—Precasting Shed.

fuel elements are passed into and out of the reactor, will be placed in five lifts. The shuttering for the soffit was suspended from steel bridges supported on the inner shielding walls.

Thermal-shielding Panels.

A small precasting works (*Fig.* 5) was provided at the site for making the concrete panels forming the thermal shield. Each panel has a rectangular frame of 9-in. steel channels, which acted as a permanent mould. The casting beds, of which there were nine in the casting shed and five in the open, comprised flat steel plates laid on a concrete slab. The plates were oiled and the frame, with a layer of mesh reinforcement welded within it, was laid on the bed and filled with concrete. The surface was smoothed by hand-trowel.

The concrete mixing plant comprised a double-bucket swivel-type batch-weigher and a pan-mixer. The panels were steam-cured under temporary covers and after twenty-four hours, when the crushing strength of the concrete was not less than 2500 lb. per square inch, the panels were lifted by a 5-ton block-and-tackle suspended from a small travelling gantry, and stood on edge outside the casting shed to mature (*Fig.* 5). The panels are of various sizes and the majority of them weigh about 3 tons each. They were lifted by eye-bolts screwed into holes in the frames.

The method of placing the panels in position inside the reactor structure is shown in *Fig.* 6. There is a small cavity, indicated by (C) in *Fig.* 4, between the face of the wall and the panels of the thermal shield; to prevent rubbish collecting in this cavity strips of rubber were laid in the horizontal joints between the panels so as to cover the gap. The strips were removed as successive panels were placed above. The panels are maintained in place by metal lugs attached to the steel frames.

Concrete Plant.

There are about 300,000 cu. yd. of concrete in the completed works for which two main concrete mixing plants were provided. One plant was near the reactor structures and the other adjacent

to the turbine-hall. There were other subsidiary concrete plants. The coarse aggregate is carboniferous limestone from the Mendip Hills. The fine aggregate is mainly sand from the Bristol Channel, with which is mixed about 10 per cent. of dune sand. In both of the main plants the aggregate was delivered to the bins over the mixers by means of inclined conveyors. Cement was delivered loose in pressure-tank wagons and stored in silos at each plant.

The plant near the reactor structures comprised two 1-cu. yd. mixers and could produce 60 cu. yd. of concrete in an hour. The concrete was discharged on either side of the plant into dumpers or into skips carried on dumpers or lorries (*Fig.* 7). The concrete for the foundations of the reactors was placed by pumps. The concrete for the walls of the reactor structures was discharged from the mixer into a lorry having a tilting steel body in which it was carried to a position near the reactor structures, where it was tipped into bottom-opening steel skips placed in a pit sufficiently large to take two 1-cu. yd. skips. The skips were lifted out of the pit by a 10-ton derrick crane having a 120-ft. jib. A pair of these cranes was provided at each reactor structure. Each crane was supported on 82-ft. high gabberds mounted on bogies on rail tracks, thereby allowing the crane to travel alongside the work in course of construction; motive power was provided by a motor on each bogie.

The concrete plant near the turbine hall (*Fig.* 8) comprised two 1-cu. yd. mixers, one of which discharged into wheeled containers and the other into a concrete pump installed in a pit below the mixers. This plant supplied concrete for the turbine hall, the pump-house, and other works in connection with the cooling-water system. About 35,000 cu. yd. of concrete are required for the turbine-hall and culverts. The basement of the turbine-hall is about 750 ft. long, 130 ft. wide and 30 ft. deep, and about 100,000 cu. yd. of material has been excavated. Concrete was pumped to the site of the pump-house, which, together with ancillary works, is contained in an excavation 163 ft. long, 125 ft. wide, and 80 ft. deep. The 18-ft. diameter access shaft to the intake tunnels is 90 ft. deep and is lined with cast-iron segments.

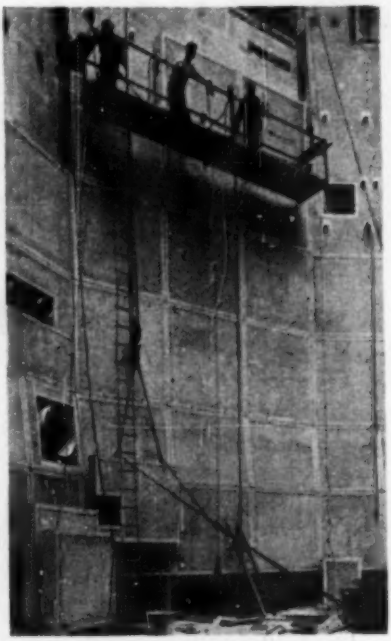


Fig. 6.

Cooling-water System.

The cooling-water system will deal with 35,000,000 gallons of sea-water per hour for the present station, but the intake and outfall works are large enough to allow for extension of the station. The intake comprises a vertical shaft about 2000 ft. from the shore, two tunnels of 12 ft. diameter for conveying the water from the shaft to the pump-house on the shore, and a parallel access tunnel of 7 ft. diameter. The tunnels were driven at a rate of about 100 ft. per week. For the shoreward half of each water tunnel steel colliery-type arches with a concrete lining were provided, and for the remainder of each tunnel precast concrete and cast-iron segments are used. The outfall works are about 2300 ft. east of the intake, and comprise reinforced concrete rectangular culverts extending 2000 ft. from the turbine-hall and discharging into a channel formed in the rocky sea-bed.

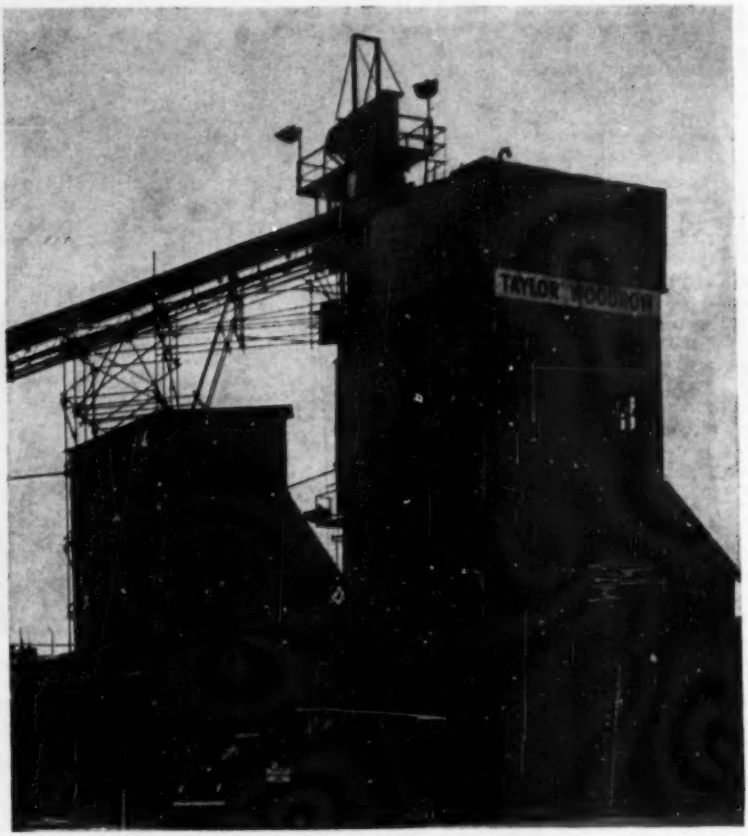


Fig. 7.—Concrete Plant near Reactors.

Intake Caisson.

The cooling-water intake structure is a caisson in the form of a reinforced concrete cylinder 90 ft. in diameter, 60 ft. high, and weighing 3900 tons, with reinforced concrete internal bracing (*Fig.* 10). The cylinder, with two surrounding rings of steel sheet-piles (B) and (D), was constructed in a dry dock. The cylinder had a false bottom of timber secured to steel lattice girders spanning between the walls at the bottom. The operations of floating the caisson out of the dock, towing, positioning, and grounding required deep water and had to be carried out in four hours on the selected day (June 20, 1959) as only in this period was the depth of water sufficient. The caisson was first drawn northwards out of the flooded dock by winches operating from a boom-defence vessel. For this movement a channel had been dredged in the foreshore ; about 30,000 cu. yd. of rock was excavated. The caisson was towed (*Fig.* 9) by an ocean-going tug through a channel marked by buoys and so into deeper water, and thence eastwards to its permanent position which is about 2000 ft. from the shore. The distance towed was about two-thirds of a mile. The caisson was then moored to anchors previously fixed in the approximate

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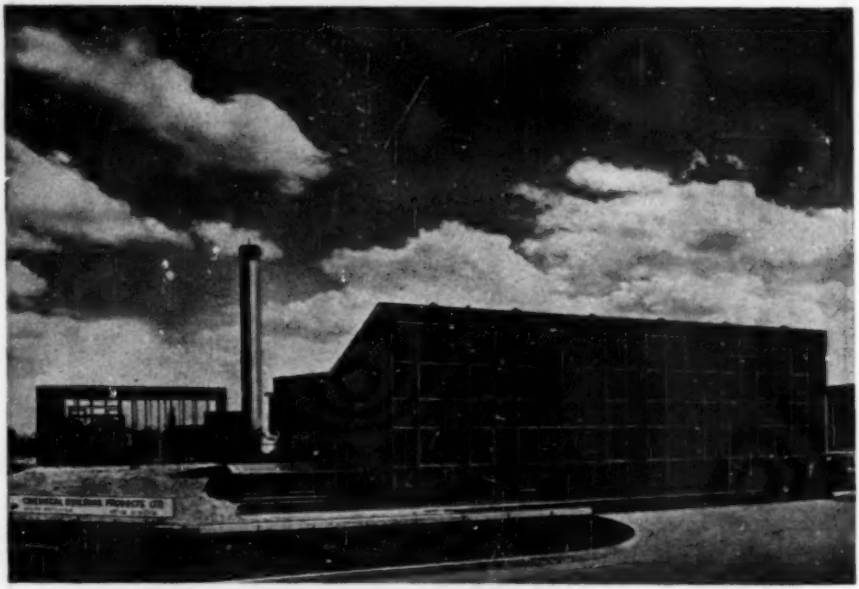

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Fig. 8.—Concreting Plant near Turbine Hall.

permanent position. The caisson and the various craft were in communication by radio-telephone throughout the operations. The caisson was manœuvred into its final position by winches on the caisson and in accordance with directions given from the shore by radio-telephone from survey stations on the shore. The operation was made difficult by the strong and variable current and the tide, which had a range of 35 ft. and was falling rapidly.

Grounding was carried out by lowering eight steel legs from the floating caisson down to the sea-bed, which had been surveyed to determine the required length of each of the legs. During the towing operations the legs, which are steel tubes, were mounted on the caisson. Each leg was lowered by an hydraulic

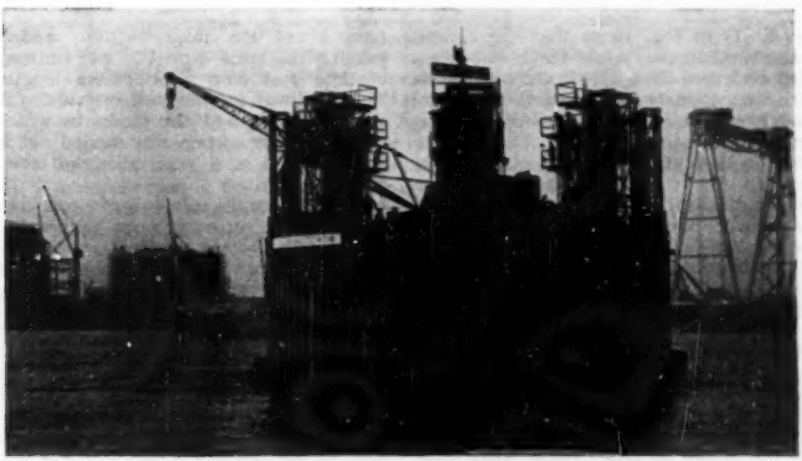

Fig. 9.—Towing Intake Caisson.

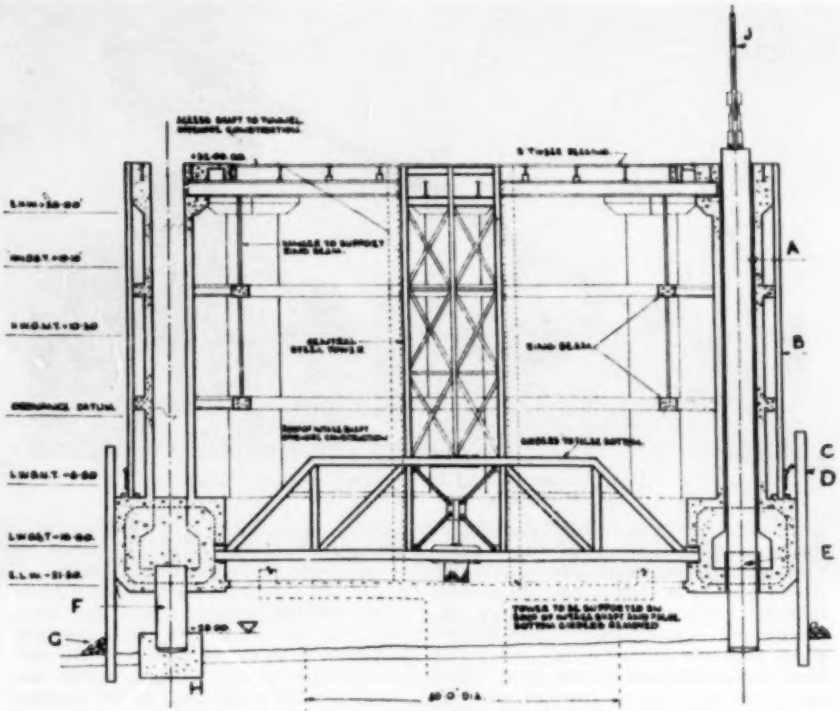

Fig. 10.—Cross-section of Intake Caisson.

jack (J) in *Fig.* 10 so that the caisson was kept vertical while the base of each leg sank into the sea-bed, and so that the load on each leg was maintained equal while the buoyancy of the caisson was reducing as the tide fell. The position of the legs when extended is shown at (A) in *Fig.* 10. When the caisson was satisfactorily supported on the legs, valves (C) were opened to admit water which flooded the interior and so made the caisson stable at all states of the tide. The operations from leaving the dock to flooding the caisson took eight hours.

The next stage was to form a cofferdam around the caisson by driving the steel sheet-piles in the lower ring (D) into the sea-bed, and to form a seal (G). Inside the cofferdam a shaft was excavated 40 ft. into the sea-bed to connect to the intake tunnels. Other work done in the dry within the cofferdam included removal of the false bottom, underpinning the spud legs (H), and burning off and removing their excess length. The length of each leg left in position is shown at (F), and the recess in which the burning operation was carried out is at (E). The steel sheet-piles will then be extracted.

Concrete for the intake works was conveyed by a ropeway over the water. Four intermediate supporting towers were provided. The tensioning device was a precast cylindrical block of concrete suspended in a shallow well on the shore.

Culverts and Cable Tunnels.

The concrete for the culverts was delivered from the mixer to the site of placing in 1-cu. yd. batches in 1¼-cu. yd. diesel-driven dumpers. The reinforcement in the culverts is shown in *Fig.* 11.

The shuttering for the inner face of

Gateway Flats, Dover

Client: Dover Corporation. Architects: Dalgliesh & Pullen, F/F.R.I.B.A. Consultants for Civil Engineering Work: S. H. & D. E. White

This important structure is supported by 1152 piles 17" in dia. and up to 30' long. The frame is of reinforced concrete throughout, and as far as practicable, beams were precast. Staircases, balcony slabs, and panels were also precast, thus effecting considerable economies in formwork costs for these members. Wind frames, of multi-bay, multi-storey, fixed portal type, are incorporated in the design to withstand the maximum exposure condition of the British Standard Code of Practice, in view of the exposed nature of the site beside the English Channel.
The Scheme is of exceptional length—1400 feet overall—and lying below Dover Castle it forms a striking addition to the historic water-front.

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Fig. 11.—Reinforcing the Culverts.

the walls and soffit of the roof of the cable tunnel from the turbine hall to the switchgear compound comprised timber sheeting on steel frames which were mounted on castors to form a travelling shutter 30 ft. in length for each of the two compartments (*Fig.* 12). The shuttering for the outer face of the side walls was of plywood attached to soldiers formed of steel channels.

Reservoir.

A reserve of water for the station is provided by a reinforced concrete covered reservoir of 2,000,000 gallons capacity which was constructed in eighteen weeks, during the winter of 1958–59. The reservoir, which is at Farringdon Hill about four miles from the station, is 180 ft. long, 120 ft. wide, and 18 ft. deep and has a central division wall and an internal valve chamber. The walls are 15 in. thick at the bottom and 9 in. at the top. The concrete roof slab is 7 in. thick and is supported on columns 12 in. square placed at 15-ft. centres. The bottom of the reservoir is 5 ft. below the original surface of the ground. Earth is

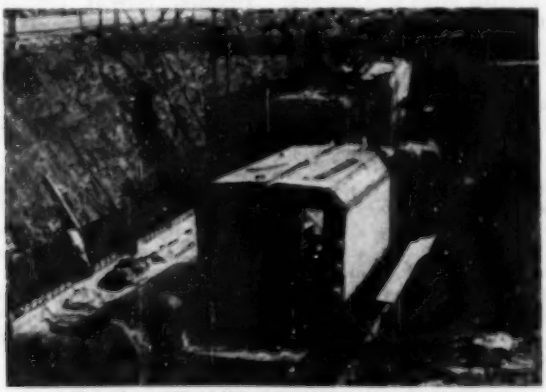

Fig. 12.—Travelling Shutters for Culvert.

banked up against the walls, a bed of stone chippings 6 in. thick is laid on the roof, and the entire area is covered with top soil and sown with grass.

The station, which is expected to be completed in 1963, is being erected for the Central Electricity Generating Board by an atomic-power group comprising The English Electric Company, Ltd., Messrs. Babcock & Wilcox, Ltd., and Taylor Woodrow Construction, Ltd., the last-named being the civil engineering and building contractors for the complete station and the reservoir.

The engineer for the reservoir was Mr. S. E. Balch, Engineer and Surveyor to the Williton Rural District Council, and the consulting engineers were Messrs. L. G. Mouchel & Partners.

The Durability of Piles in Sea-water.

THE twentieth report* of the Sea Action Committee of the Institution of Civil Engineers was published recently by the Department of Scientific and Industrial Research. Some of the conclusions from the tests, which have extended over a period of twenty-three years, are as follows.

The general conclusion is that the primary cause of deterioration of reinforced concrete piles is corrosion of the reinforcement and not disintegration of the concrete from the chemical or physical effects of the salts in sea-water. The salts nevertheless influence corrosion, since this becomes progressively greater as the resistance of the cement to direct attack decreases. The cement content and the thickness of the cover of concrete over the reinforcement were of major importance in determining the durability of piles. With a 1 : 2½ mixture and a cover of 2 in., no cracking occurred with any cement either in the first ten years of controlled exposure or in the subsequent thirteen years of uncontrolled exposure. This result applied also to the 1 : ½ : 4½ pozzolana-concrete which in actual Portland-cement content corresponds to 1 : 5. No corrosion of the steel was found on breaking the piles. With the more usual mixture of 1 : 5 by weight and with 2-in. cover, cracking occurred to a greater or lesser extent with Portland cement in the first ten years and became more accentuated in the subsequent thirteen years. Similar piles made with high-alumina, Portland-blastfurnace, and trass–Portland cements (corresponding to a 1 : 9 mixture in Portland-cement content alone) were entirely free from cracking after ten years. After the subsequent thirteen years piles containing high-alumina cement and trass–ordinary Portland cement mixture only remained uncracked, and on breaking these piles some corrosion of the steel was found in about half of both the trass–Portland cement and the high-alumina cement piles.

With 1-in. cover and 1 : 2½ concrete, only the piles made with high-alumina and Portland-blastfurnace cements remained free of cracks after ten years and only with high-alumina cement after the subsequent thirteen years. Some corrosion of the steel over lengths of a few inches was, however, observed in the latter piles. With 1 : 5 concrete, only high-alumina cement concrete remained uncracked after ten years, but cracked later.

The results show that a 1 : 5 by weight Portland cement (ordinary or rapid-hardening) concrete, and 2-in. cover is insufficient to guarantee protection against corrosion of the steel above low-tide level and that a richer mixture or thicker cover is required. Over a period of ten years, a 2-in. cover of 1 : 5 concrete sufficed to protect the reinforcement from corrosion with Portland-blastfurnace and trass–Portland cements, but after twenty-three years some of the piles with the former cement had cracked and also some of those with trass cement. High-alumina cement gave the best performance up to the twenty-three years, but the behaviour of this cement can be affected if a high temperature is reached within the concrete.

* "The Durability of Reinforced Concrete in Sea-water." (London: H.M.S.O. 1960. Price 4s.)

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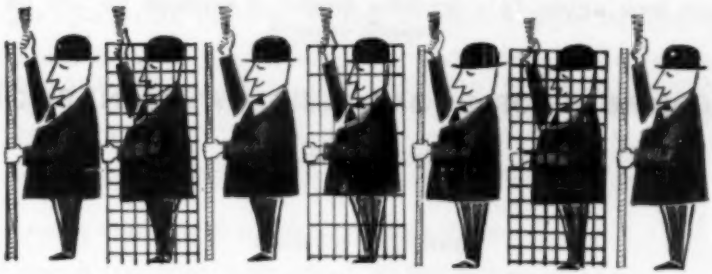

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Comparison of Analyses of Helical Stairs.

PROFESSOR A. R. CUSENS of SEATO Graduate School of Engineering, Bangkok, comments as follows on the article entitled "Comparison of Analyses of Helical Stairs" by V. A. Morgan, which was published in the number for March, 1960. The comments apply to the analysis of a flight of helical stairs with fixed ends.

The terms δm and M in equation (10) should be M_v in each case, and I_1 and I_2 in equation (11) should be $\frac{db^3}{12}$ and $\frac{bd^3}{12}$ respectively; for this purpose the cross-section of the waist is assumed to be rectangular. I_1 is taken about a horizontal axis, and I_2 is about an axis normal to the slope of the stairs.

The coefficient K_2, in the expression for the polar moment of inertia, is shown in "Theory of Elasticity" by S. Timoshenko and J. N. Goodier to be less than 0·3 (say, 0·296 by interpolation) when $\frac{d}{b} = 6$. Therefore $\frac{CJ}{EI_1} = 1{\cdot}42$ and $\frac{CJ}{EI_2} = 0{\cdot}0395$ instead of 1·49 and 0·0415 respectively.

Equation (13), when corrected and simplified, reads

$$\left\{\frac{CJ}{EI_1}\left[wR_1^2 - K(M_v + wR_1^2) + \frac{HR_2}{2}\tan\phi\left(\frac{\theta^3}{3} - \frac{\theta^2\sin 2\theta}{2} - 2K\right)\right]\right.$$
$$+ s\left[K(M_v + wR_1^2) + wR_1R_2(\theta^2\sin\theta + 2n) + \frac{HR_2}{2}\tan\phi\left(\frac{\theta^3}{3} + \theta^2\frac{\sin 2\theta}{2} + 2K\right)\right]$$
$$+ \left(s - \frac{CJ}{EI_2}\right)\left[m(M_v + wR_1^2) + nwR_1R_2 + 2KHR_2\tan\phi\right]$$
$$\left. + mHR_2\cos^2\phi\left(\tan\phi + \frac{CJ}{EI_2}\cot\phi\right)\right\}_0^{\frac{1}{2}\beta} = 0 \text{ (when } \theta = \tfrac{1}{2}\beta).$$

Since θ is variable, equation (13) applies between the limits of zero and $\frac{1}{2}\beta$. These limits also apply to equation (12).

These revisions do not affect greatly the numerical values of H and M_v as calculated in the article.

MR. V. A. MORGAN replies that he is in agreement with the foregoing revisions and confirms that they do not materially affect the magnitudes of the forces and moments or the curves given in *Fig.* 3 of the article. The more accurate values are slightly less than those calculated.

A Proposed Formwork Discussion Group.

AT an informal meeting held recently in London and attended by about 150 representatives of civil engineering and building contractors it was proposed that a "formwork discussion group" should be formed. The object of the discussions would be to fill the gap between the preparatory work carried out at technical colleges and practical application in matters relating to shuttering for concrete. Engineers and others interested in this proposal should communicate with Mr. W. F. Whyman, Kwikform, Ltd., East Road, Upper Wickham Lane, Welling, Kent.

A lecture was given at the meeting by Mr. H. R. Gill, author of "Concrete Formwork Designer's Handbook."

Fig. 1.

A Dome at the Tel-Aviv Jubilee Exhibition.

One of the most interesting structures at the Tel-Aviv Jubilee Exhibition, in Israel, is the dome of the pavilion for the Municipality of Tel-Aviv (*Fig.* 1). The dome is the shape of a cap of a sphere intersected by six inclined planes. The diameter at the supports is 43 m. (about 149 ft.) and the height is 7·08 m. (about 23 ft. 3 in.). The thickness of the dome is 3¼ in. at the crown, 10¼ in. at the entrances and 1 ft. 3 in. at the supports.

ment in a radial direction to take place during tensioning of the tie-beams and to eliminate the effects of shrinking of the concrete and movements due to changes of temperature. Two days after concreting the dome a reduction, due to shrinking, of 1 in. in the diameter was observed.

During tensioning of the ties an upward deflection of 1 in. at the crown was observed. This movement freed the shuttering which was then easily removed. The

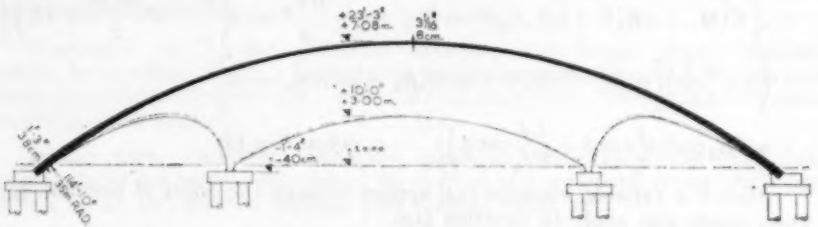

Fig. 2.—Cross-section of the Dome.

The width of the "legs" of the dome at the supports is 1 ft. 10 in. A cross-section of the dome is shown in *Fig.* 2.

Each leg is supported by a pile cap on four bored piles of 80 cm. (about 2 ft. 8 in.) diameter. The six pile caps are connected underground by six prestressed tie-beams 16 in. wide and 2 ft. thick and each supported by two bored piles equally spaced along the length of the beam. Each beam is prestressed by eight cables each having twelve wires of 5 mm. diameter. The wire has a breaking strength of 120 tons per square inch.

The vertical load on each pile cap is about 100 tons and the tension in the tie-beams, 180 tons. Five of the pile caps are on rollers and one is fixed. Each cap is in two parts (*Fig.* 4) to enable move-

compressive stresses at the crown were about 6 kg. per square centimetre (85 lb. per square inch) and the maximum shearing stress at the supports was about 6 kg. per square centimetre (85 lb. per square inch). To enable the reinforcement to be fixed securely in position, and to enable the workmen to move freely about the shuttering, a welded fabric of 8-mm. wires at 6 in. centres was laid on concrete spacers on the shuttering. The main reinforcement was bound to this fabric.

The concrete was a 1 : 2 : 4 mixture using Portland cement and was ready mixed. It was decided to concrete the dome continuously and the 250 cu. m. (327 cu. yd.) of concrete were placed in seventeen hours. Six gangs of workmen

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Fig. 3.—Concreting the Dome.

started one at each support and worked towards the crown. The concrete was delivered in mixing trucks and lifted to the top of the scaffolding (*Fig.* 3) along which it was taken to a distributing plant at the crown. From the crown the concrete was distributed through troughs to various parts of the dome.

The marks of the wooden shuttering were left on the surfaces of the concrete to which the only finish applied was a white cementitious wash on the top of the dome.

The structure was designed as a circular dome supported by a ring beam which resists the forces of the six inclined legs. Secondary bending moments were determined according to the theory of shells.

The architect is Mr. Aril Elhanani, and the consulting engineer is Mr. M. A. Vardimon. The construction was supervised by the City Engineer, Mr. M. Amiaz. The contractors were Messrs. Solel Boneh, Ltd. The information in this article was supplied by Mr. J. S. Cohen.

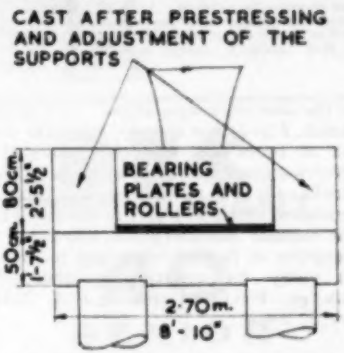

Fig. 4.—Pile-cap and Roller Bearings.

Publications Received.

"Preliminary Publication of the Sixth Congress of the International Association for Bridge and Structural Engineering." (Zürich: Verlag Leeman. 1960. Price 70 Swiss francs.)

The texts of several papers include six on the properties of materials, ten on the development of methods of calculation, twenty-three on metal structures, five on reinforced and prestressed concrete bridges, four on the safety of concrete structures, and three on precast concrete structures.

"Mechanical Properties of Structural Materials at Low Temperatures." By M. McClintock and H. P. Gibbons. [United States Department of Commerce. National Bureau of Standards. 1960. Price 1.50 dollars (plus 25 per cent. for postage outside U.S.A.) from Government Printing Office, Washington, D.C.]

A series of graphs compiled from the literature on the subject and relating mainly to non-ferrous metals and some steel alloys and plastics.

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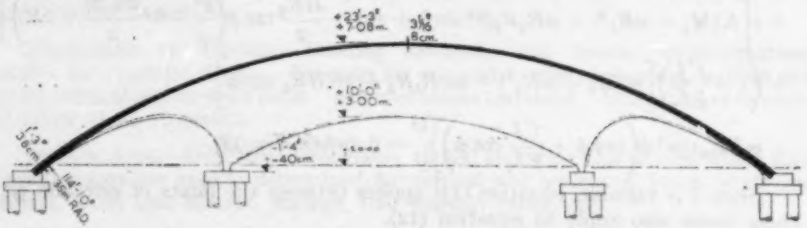

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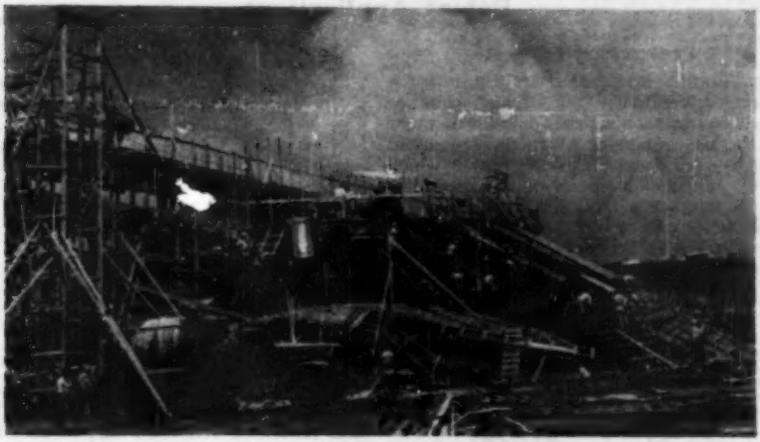


Fig. 3.—Concreting the Dome.

started one at each support and worked towards the crown. The concrete was delivered in mixing trucks and lifted to the top of the scaffolding (*Fig.* 3) along which it was taken to a distributing plant at the crown. From the crown the concrete was distributed through troughs to various parts of the dome.

The marks of the wooden shuttering were left on the surfaces of the concrete to which the only finish applied was a white cementitious wash on the top of the dome.

The structure was designed as a circular dome supported by a ring beam which resists the forces of the six inclined legs. Secondary bending moments were determined according to the theory of shells.

The architect is Mr. Aril Elhanani, and the consulting engineer is Mr. M. A. Vardimon. The construction was supervised by the City Engineer, Mr. M. Amiaz. The contractors were Messrs. Solel Boneh, Ltd. The information in this article was supplied by Mr. J. S. Cohen.

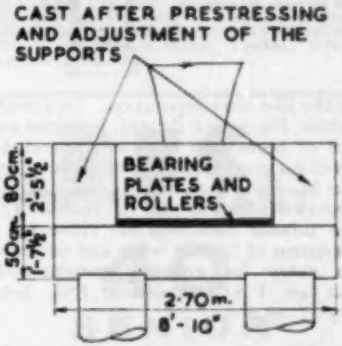


Fig. 4.—Pile-cap and Roller Bearings.

Publications Received.

"Preliminary Publication of the Sixth Congress of the International Association for Bridge and Structural Engineering." (Zürich: Verlag Leeman. 1960. Price 70 Swiss francs.)

The texts of several papers include six on the properties of materials, ten on the development of methods of calculation, twenty-three on metal structures, five on reinforced and prestressed concrete bridges, four on the safety of concrete structures, and three on precast concrete structures.

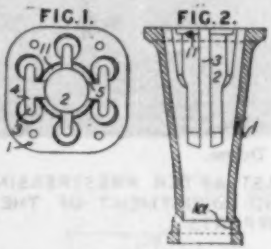
"Mechanical Properties of Structural Materials at Low Temperatures." By M. McClintock and H. P. Gibbons. [United States Department of Commerce. National Bureau of Standards. 1960. Price 1.50 dollars (plus 25 per cent. for postage outside U.S.A.) from Government Printing Office, Washington, D.C.]

A series of graphs compiled from the literature on the subject and relating mainly to non-ferrous metals and some steel alloys and plastics.

Patent Applications.

Prestressed Concrete.

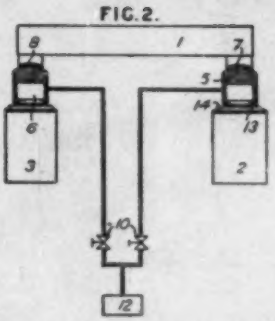
ANCHORING means for securing in the stressed condition reinforcing wires or rods comprises a tapered tubular member 1 having a main aperture 1a at one end to receive bunched wires or rods which are connected by bores 3 and slots 4, 5 to a number of subsidiary apertures at the other end grouped around the main aperture 2 so that individual wires may be located therein and secured by a collet



or the like after separation. In a modification, *Fig.* 6 (not shown), a greater number of bores and slots are provided to receive a greater number of wires. Centre blocks, *Figs.* 9–13 (not shown), may be provided which fit into the recesses 11 on the tubular members and allow for the reception of further wires and to provide an entry for grouting material.—No. 822,540. P.S.C. Equipment, Ltd. February 7, 1958.

Adjustable Foundations.

In order to avoid stresses arising in building foundations due to local movements or subsidences in the supporting ground the foundation 1 is carried on supporting members 5 with convex spherical surfaces 7 engaging spherical sockets 8 on the foundation 1. The members 5 are also adapted to slide horizontally on plates 14 secured to support pedestals 2, 3 and are provided with hollow chambers 6 which can be filled with liquid under pressure supplied from a pump 12 through valves 10 to compensate a proportion of the load to allow such sliding. Sealing



rings 13 are provided in the chambers 6. An hydraulic lifting device is preferably also combined with each support member 5 so that the foundation may be quickly restored to horizontal position after any shift has occurred.—No. 825,167. Maschineufabrik Augsburg-Nurnberg A.G. March 12, 1957.

Strength of Concrete containing Pulverised-fuel Ash.

THE article entitled "Strength of Concrete containing Pulverised-fuel Ash" in the October number of this journal was extracted from the "Journal of the American Concrete Institute", June 1958.

Change of Name and Address.

CONSIDÈRE CONSTRUCTIONS LTD., which is one of the pioneer firms of the reinforced concrete industry in Great Britain and has occupied the same premises in Westminster for forty years, have moved to 66/71 Wilton Road, London, S.W.1. The building in Victoria Street which the firm previously occupied is being demolished.

The name of the firm has also been changed to Considère & Partners, Ltd., which is thought to be more appropriate to the business of consulting engineers specialising in the design of reinforced concrete and structural steelwork.

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 5d. a word : minimum, 12s. Situations Vacant, 6d. a word : minimum, 15s. Other miscellaneous advertisements, 6d. a word : minimum, 15s. Displayed advertisements, 40s. per column inch. Box number 1s. extra.

Advertisements must reach this office, 14 Dartmouth Street, London, S.W.1, by the 23rd of the month preceding publication.

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SITUATIONS VACANT. Reinforced concrete detailers required by LEONARD & GRANT in their South London drawing offices. Only top men should apply. High salaries. Interview by appointment. Telephone Tate Gallery 8496.

SITUATIONS VACANT. Reinforced concrete designers and detailers for office near Swiss Cottage, London, N.W. Three weeks' holiday after three years, excellent prospects, and friendly atmosphere in small firm. Minimum experience two years. Telephone Maida Vale 7890.

SITUATIONS VACANT. Consulting engineers have vacancies for the following: Reinforced concrete designers with A.M.I.Struct.E.; Designer-detailers with H.N.C. or similar; Detailers with minimum two years' experience. The positions are permanent, with opportunity for advancement. Modern office and pleasant conditions. Salaries in accordance with experience. BYLANDER, WADDELL & PARTNERS, 169 Wembley Park Drive, Wembley.

SITUATIONS VACANT. A vacancy exists for a reinforced concrete designer or an experienced designer-detailer and for an experienced detailer-draughtsman. Good prospects. Varied and interesting work. Luncheon vouchers. 5-days' week. Apply, with particulars of experience and salary required, to JOHN F. FARQUHARSON & PARTNERS, 34 Queen Anne Street, London, W.1.

SITUATION VACANT. Qualified young engineer-designer required by London consulting engineers. Good background of drawing office and other experience in reinforced concrete work is essential. Work will be primarily in drawing office, but successful applicant will assist executive with technical administration work, organisation, correspondence, accounts, estimating, etc. Good prospects for smart and alert applicant who can take things as they come. Starting salary about £1250. Applications with full details in confidence to BOX 4706, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Designer-detailers required preferably qualified but minimum standard H.N.C. with several years' drawing office experience in reinforced concrete. Age range 25–40 and able to work on own initiative. Applications in writing to JOHN LIVERSEDGE & ASSOCIATES, 42 Portland Place, London, W.1.

SITUATIONS VACANT. Air Ministry Works Designs Branch requires in London structural engineering designer-draughtsmen for reinforced concrete or structural steelwork of all types. Applicants must have adequate training and several years' experience. Some site experience and possession of recognised technical qualification an advantage. Financial assistance and time off may be allowed for recognised courses of study. Promotion and pension prospects. 5-days' week with 18 days paid leave per year initially. Salary ranges from £805 (at age 25) to £980 per annum. Commencing salary dependent on age, qualifications and experience. Applicants, who must be natural born British subjects, should write (quoting Order No. Kings Cross 3745) to AIR MINISTRY W.G.d., Lacon House, Theobalds Road, London, W.C.1, or to any Employment Exchange, giving age, details of training, qualifications, full particulars of former posts held and copies of any testimonials. Candidates selected will normally be interviewed in London and certain expenses reimbursed.

SITUATION VACANT. Structural or civil engineering designer required for Head Office appointment. Experience in reinforced concrete design, foundations and industrial structures an advantage. Permanent position with good salary, bonus scheme, and entry into superannuation scheme after 12 months' service. Good opportunity for qualifying as A.M.I.C.E. and A.M.I.Struct.E. Write, giving age, qualifications and details of experience, to: THE MITCHELL CONSTRUCTION COMPANY LTD., Wharf Works, Peterborough.

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SITUATIONS VACANT. Ove Arup & Partners have vacancies in their London, Manchester, and Edinburgh offices for engineers, designer-detailers, and detailers experienced in reinforced concrete, for work on a variety of interesting projects. Interviews will be arranged at the appropriate office as convenient. Applications, giving brief details of previous experience, qualifications, etc., should be sent to the address below. Salaries offered will be according to ability and experience, and additional benefits include luncheon vouchers, bonus and voluntary superannuation scheme. OVE ARUP & PARTNERS, 13 Fitzroy Street, London, W.1.

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SITUATION VACANT. Consulting Engineers, London and Surrey, have vacancy in their Haslemere office for experienced reinforced concrete designer-detailer, salary £1,000 to £1,200 per annum according to experience. A very attractive modern detached house with garage is available to the successful applicant at nominal rental. Apply Box 4715, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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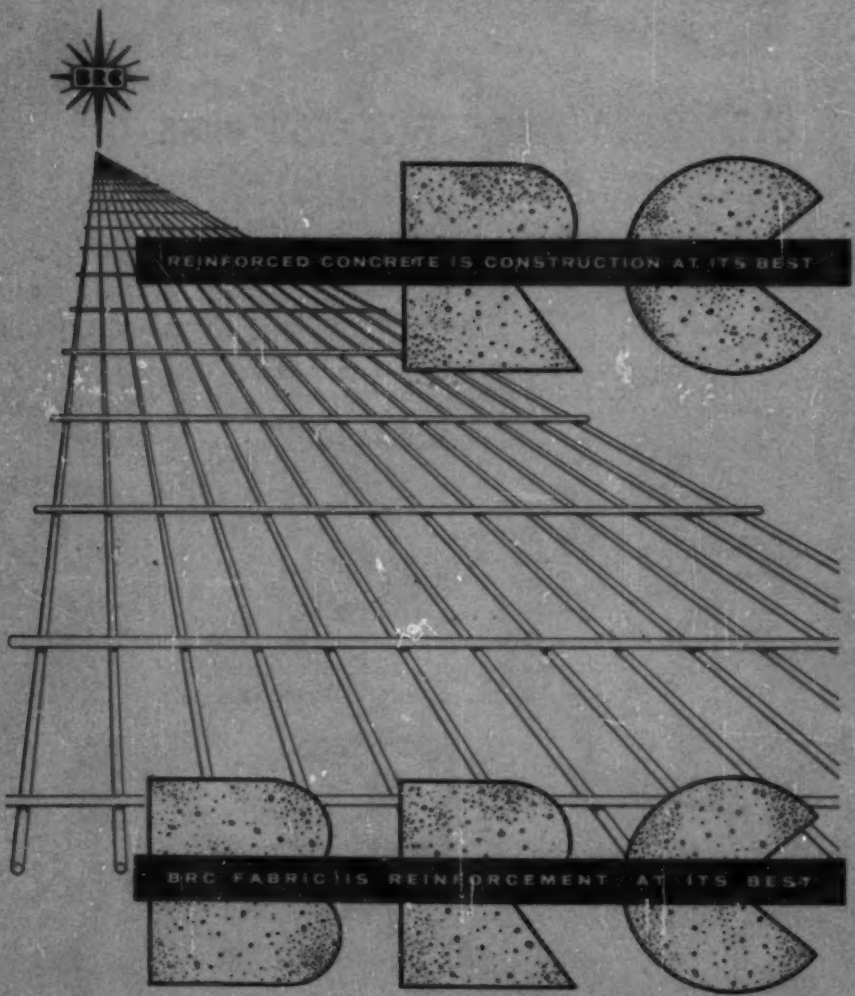

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Published by Concrete Publications Ltd., 14 Dartmouth Street, London, S.W.1. Telephone: Whitehall 4581
Printed by Butler & Tanner Ltd., The Selwood Printing Works, Frome and London